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SNETTISHAM PROJECT ALASKA. FIRST STAGE DEVELOPMENT. DESIGN MEMO--ETC(U)

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DESIGN MEMORANDUM NO. 7.

GENERAL DESIGN MEMORANDUM

...RT

(12) 306p.

(11) OCTOBER 1965

U.S. ARMY ENGINEER DISTRICT, ALASKA

CORPS OF ENGINEERS

COPY

(1)

DEPARTMENT OF THE ARMY
U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
P. O. BOX 7002
ANCHORAGE, ALASKA 99501

IN REPLY REFER TO
NPAEN-PR-R

13 November 1965

SUBJECT: Snettisham Project, Alaska; Design Memorandum No. 7, General Design Memorandum

TO: Division Engineer
North Pacific Division

1. Transmitted under separate cover are 16 copies of subject design memorandum, each consisting of two volumes.

2. Estimated project cost of the first stage of the Snettisham Project is $40,300,000. Benefit-to-cost ratio based on 100-year project life is 2.20 to 1. Preliminary estimated cost of the second stage development is $13,000,000. When combined with the more accurately determined cost of the first stage of the project, the estimated project cost of the total development is $53,300,000. Benefit-to-cost ratio for the total project based on 100-year project life is 2.57 to 1.

3. Because of the desirability of awarding one contract to cover all excavation work, and the heavy design load associated with such procedure, your early approval of the General Design Memorandum is requested. In the interim, this office proposes to continue final design studies in accordance with the plan recommended therein.

Clare F. Farley
CLARE F. FARLEY
Colonel, Corps of Engineers
District Engineer

1 Incl (under sep cover)
as (16 cys of 2 volumes ea)

COPY

NPDEN-TE (13 Nov 65) 1st Ind
SUBJECT: Snettisham Project, Alaska; Design Memorandum No. 7, General Design Memorandum

North Pacific Division, Portland, Oregon, 4 January 1966

TO: Chief of Engineers, ATTN: ENGWE-EZ

1. The design memorandum is recommended for approval subject to the comments which follow. Some of the material is presented in considerable detail for a general design memorandum, particularly that dealing with geology, transmission lines, and stability analyses. Studies in these areas are continuing and the following comments will be used in the preparation of the appropriate feature design memorandums.

a. <u>Section 5</u>.

(1) This section presents a very conservative approach to the treatment required to stabilize the foundation. Paragraph 5.13 assigns a shear strength of only 25 and 15 psi to the joint surfaces. Considering the irregularity of the joint surfaces and the load applied if prestressing is to be used, a much higher shear strength possibly in the order of 150 to 200 psi appears appropriate. The inclusion of foundation prestressing is satisfactory for estimating purposes but a decision on the use of prestressed tendons should be withheld until additional foundation explorations confirm the need for this type of treatment.

(2) The desirability of area grouting in the foundation area in connection with Plan B (prestressed foundation) is questionable because of the adverse effects on natural drainage. More consideration should be given to the need for area grouting if prestressing is included in the final plans by a study of six inch cores across the joints and the bore hole photographs. If these indicate that the rock matches perfectly, the area grouting should be deleted. Curtain grouting should be the same for all plans considered. With the apparent openness of joints as indicated by water losses during drilling, a line of drainage holes as planned in paragraph 4.18 in the grouting and drainage gallery along with the daylighting of the joints downstream should provide adequate drainage.

(3) The further investigations discussed in paragraph 5.19 should not preclude continuation and possible completion of the concrete dam design in the interim. It can be assumed that a satisfactory foundation will be provided and whether it is at Elevation 785 or 800 ± should make little difference except for quantities.

NPDEN-TE (13 Nov 65) 1st Ind 4 January 1966
SUBJECT: Snettisham Project, Alaska; Design Memorandum No. 7, General Design Memorandum

b. Paragraphs 5.10 and 5.14. Use of the average slope of the undulations in the low angled joint planes is believed overly conservative in analyzing the foundation strength. Resistance to sliding increases rapidly as the slope increases. Undulations steeper than about 60 degrees have a shear friction factor of safety greater than 4 without regard to shearing strength of the rock. Thus any tendency to slide is resisted by the steeper irregularities by shear through the rock.

c. Paragraph 5.13. Prestressing tendons cannot resist shear in this application because of overstress of rock or concrete in side bearing.

d. Paragraphs 5.19 and 6.12.

(1) As indicated above, final determination on the foundation for the dam should be deferred at least until the time of submittal of the specific design memorandum.

(2) Although prestressing has been used to strengthen existing dams, its use accompanied with a reduction in the dam cross section for this project is not recommended in the interest of safety.

e. Paragraph 6.06. The basis of selecting 1.5 feet of freeboard was not given. This amount appears inadequate and a freeboard of approximately 3 feet is recommended.

f. Paragraph 6.09. The unusually low design shear values used in the stability analyses should be reconsidered on the basis of additional subsurface studies as indicated in paragraph a(1) above.

g. Paragraph 6.12. The foundation for the maximum section will be at approximate Elevation 810 for Plans A and B and 785 for Plan C. A standard gravity section should be designed for the maximum height. This section will be satisfactory for any foundation elevation finally selected. The final concrete quantities can be adjusted at any time prior to advertisement to fit the foundation grade finally selected.

h. Paragraph 6.14. The need for concrete paving on the invert and pneumatically-applied mortar on the side should be based on an economic analysis.

i. Paragraph 7.05. Reference to timber sets should read "steel sets".

j. Paragraphs 7.06 and 8.06. The amount of rock bolting is considered conservative for the apparent good rock that is expected. However, it is satisfactory for preliminary estimating. Rock bolting should be based on requirements during construction on the basis of safe practices.

k. Paragraphs 7.11 and 7.13. Little is believed to be gained in designing the tunnel linings from the rock mechanics tests described in these paragraphs. The tests should be held to a minimum or eliminated.

l. Paragraphs 7.10 and 8.14. Minimum penstock thicknesses should be based on buckling loads from external pressure. These may be determined by the methods of Vaughan or Amstutz. To facilitate fabrication Type A-537 steel with a yield point of 50 ksi should be considered in lieu of Type A-517. Allowance for support from surrounding rock cover can be provided by designing the penstock liner for full internal pressure including dynamic head at 80% yield strength.

m. Paragraph 8.12. Studies conducted by this office show the optimum diameter for the Long Lake penstock to be 7.5 feet and for Crater Lake penstock to be 6.75 feet. Selection of penstock diameters will be resolved prior to submittal of the powerplant preliminary design report.

n. Section 9. An independent study conducted by this office confirms the selection of the conventional surface powerhouse. This study differed from that given in the report by inclusion of additional variables:

(1) Use of a different optimum penstock diameter for each of the four locations studied varying from 7.5 to 9.0 feet for Long Lake and 6.75 to 7.75 for Crater Lake.

(2) Different WR^2 values and crane sizes for each study.

(3) Use of a 15 percent contingency for aboveground work and 20 percent for underground work.

On this basis the total capitalized cost including head loss for the recommended plan is $11,469,000 compared to a cost for an underground powerhouse sited to provide the shortest connection between surge tank and powerhouse of $12,859,000.

o. Paragraphs 10.05 and 10.06. Reference is made to the attached copy (Incl. 3) of NPAEN-PR letter dated 28 October 1965, subject: "Criteria

D

for Establishment of Plant Capacity, Snettisham Project, Alaska". The referenced letter requests advice concerning sizing the generators to suit Bureau of Reclamation policy of not marketing capacity in the 15 percent generator overload range. We do not recommend sizing the generators to suit the marketing policy for the reason that the overload capacity is continuous and as such is marketable. However, we do recommend an increase in the generator rating for other reasons. The gross operating head associated with Long Lake will range from 720 to 895 feet and for Crater Lake it will range from 828 to 1,022 feet. It has been requested by the Bureau that special interconnections be provided at the powerhouse so that the turbines will operate from either lake in which event the operating head on the turbine will be from 720 to 1,022 feet. The arrangement of Unit 2 provides for operating from either lake. The turbines for Units 1 and 2 will be sized and matched with generators to provide dependable capacity at minimum pool or 720 feet gross head. For heads greater than 720 feet the generator will limit the plant output which will require the turbine to be operated at lesser gate openings over the head range resulting in correspondingly decreased efficiencies. A larger generator will provide greater flexibility and permit the turbine to operate at correspondingly higher efficiencies as well as provide secondary capacity which has some benefit although we have no method for its evaluation. These intangible assets have been recognized at other projects and at Dworshak the generator size was arbitrarily increased by 10 percent. In view of the above it is recommended that the nameplate ratings of the generators for Units 1 and 2 be increased 15 percent or to 26,000 KVA at 0.9 power factor, with capability of operating continuously at 115 percent rated KVA. No increase in cost of equipment other than generators is anticipated by this change.

p. Section 11. Costs and real estate requirements for helicopter pads for servicing the transmission line should be included.

q. Paragraph 11.06. Reference to Plate 3 in second sentence should be Plate 1.

r. Paragraph 12.10. The Bureau of Reclamation in Exhibit 1.1 has requested access to the dam for wheeled vehicles and is opposed to maintaining a tramway. Accordingly, a permanent project road extending to the damsite should be considered and included as a separate contract bid item. Necessary transfer of feature account from Construction Facility to 08, Roads should be accomplished prior to the next budget submission.

s. Paragraph 12.11. Approval of the Resident Engineer's office should be reserved until staffing is known and a layout has been submitted.

Normal space is 60 square feet per person. The proposed building with 3,200 square feet appears to exceed this amount considerably.

t. Paragraph 13.01. The principles of good architectural design quoted in this paragraph have not been exhibited by Plates 31 through 39 which depict building concepts lacking in originality and continuity in character between the various structures. Intensive architectural design effort and close coordination between the various design offices are required to attain a unity of design in the vicinity of the powerhouse.

u. Paragraph 16.06. The last sentence is no longer applicable because of deferment of construction start for the project.

v. Section 17. Scheduling and funding given in this section are no longer applicable

w. Paragraphs 17.04 and 17.05. It is essential that the work indicated as Contracts A-9 and B-9 be accomplished in one contract to obtain the benefit of the most economical and practicable operation. The tolerances prescribed for tunnel excavation and foundation excavation as shown are more respected by the contractor if he is responsible for the concrete that replaces overbreak. The overlap of contracts as now shown in such a restricted area is highly undesirable. Furthermore, the Army Audit Agency has recently been critical of the use of multi-contracts for project construction.

x. Paragraphs 20.10 and 20.12. The District Engineer will be requested to justify the increases in E&D and S&A costs which are reported in these paragraphs.

y. Paragraph 21.06. New power values are being developed by the Alaska District pursuant to 1st Indorsement, ENGCW-EZ to basic letter from this office, subject: "Snettisham Project, Alaska; Design Memorandum No. 3, Selection of Plan of Development", dated 29 July 1965. These revised power values will be included in the FY 67 Congressional submission.

z. Paragraph 21.07.

(1) The explanation of the 61 percent increase in benefits presented in this paragraph is inadequate. The explanation was covered in an exchange of correspondence between this office and the Alaska District in letter NPDEN-TE, subject: "Snettisham Project, Alaska; Design Memorandum No. 3, Selection of Plan of Development", dated 7 June 1965 and 1st Indorsement thereto, copy attached.

F

(2) Total benefits of $6,119,000 given in paragraph 21.07 do not agree with the $6,026,000 total given in paragraph 21.08.

aa. Figure 20. Schedule is no longer applicable.

bb. Exhibit 1.1. The foot bridge to reach the left abutment of the dam as requested by the Bureau of Reclamation has not been indicated. This should be shown and costs therefor should be included. The foot bridge could possibly be located across the stream downstream from the dam.

cc. Plates.

(1) Preparation of original tracings for plates should assure legibility of the report-size reproductions.

(2) Many of the plates and figures do not conform to paragraph 10c, EM 1110-2-1002 nor standards of the International Commission on Large Dams which require that general plans, elevations and cross sections of hydraulic structures be oriented so that the direction of flow of water is from the top to bottom of the sheet or from left to right on the sheet. Drawings for future reports and contract documents should conform to these requirements.

dd. Plates 15 and 16. Depth for the drainage and grout holes for Plan A do not extend to the same depth as for Plan C. Both plans are subject to the same reservoir pressures and have the same drainage requirements so they should be the same.

ee. Plate 14-A. The purpose of the calculations shown on this sheet is not clear. Since friction is independent of area of contact ($F = \mu N$), there is no need to assume that the load on the joint plane is supported at two points. In order to determine the sliding resistance owing to friction on the undulations and to the weight on an inclined surface, it is necessary only to sum the forces acting normal to a plane defined by angle ($\theta \pm \alpha$) with respect to a horizontal plane and extracting the horizontal resistance. This summation gives a result differing somewhat from the equation of average angles shown on the drawing and is as follows:

$$S.F. = \frac{\Sigma V}{\Sigma H}\left[\tan(\theta \pm \alpha) + \tan\phi \cos^2(\theta \pm \alpha)\right] + \tan\phi \sin(\theta \pm \alpha)\cos(\theta \pm \alpha)$$

ff. Plates 24, 25 and 26. The upstream faces of the spillway sections and nonoverflow sections should lie in the same plane using

the same batter. The fact that the spillway ogee will be in a different plane from the nonoverflow downstream face will be masked by the training walls. The upstream lip on the spillway ogee should not be required.

gg. Plate 28. Although details have not been developed for this report, the intake works should be more generously proportioned and concrete members should be made more massive. Consideration should be given to placing a divider wall behind the trashrack to separate the power tunnel inlet and the reservoir emptying inlet. If this were done, wider spacing would be used for the trashrack for the latter and could consist of reinforced concrete beams. The need for an emergency fixed wheel gate for the outlet tunnel should be considered further since this tunnel will rarely be used when the project is in operation (draining lake only).

hh. Plate 29. Provide a man door on the penstock in the access vault to permit inspection without removing the 8-foot length of removable penstock.

ii. Plate 31 and Paragraph 13.05. Powerhouse operations would be improved if the office and machine shop were made part of the powerhouse building.

jj. Hydraulic Design. The feature design memorandums for this project involving hydraulic features should include a section in the text covering hydraulic design. The following comments should be considered in the appropriate feature design memorandums:

(1) Hydraulic design criteria including preliminary rating curves for the diversion tunnel to be used both for draining the lake and diversion during the active construction period should be included. Similar information should be provided for the outlet tunnel.

(2) Bubbler-type gages for both pool and tailwater gages should be considered.

(3) The outlet control structure as now shown appears to lie in the path of flow downstream from the spillway. Diversion of the flow or relocation of the outlet structure may be required.

(4) On Pertinent Data Sheet 2 the term "deck elevation" or "top of the dam" should be used rather than "crest of dam".

H

NPDEN-TE (13 Nov 65) 1st Ind 4 January 1966
SUBJECT: Snettisham Project, Alaska; Design Memorandum No. 7, General Design Memorandum

(5) Operation of the combination of the emergency butterfly valve, transition and vertical bend in the power penstock shown on Plate 29 could produce damage to penstock. Consideration should be given to incorporating and the full transition from 9 feet to 7 feet in the vertical bend and installation of a 9-foot diameter butterfly valve. This combination should minimize the head losses in this section of the penstock. The air vent mentioned in paragraph 8.09 is not shown on the downstream side of the valve on Plate 29.

(6) Consideration should be given to other types of outlet control gates i.e., slide gate with offset guides to prevent impingement of the jet. Criteria for selection of type of outlet control gate should be provided.

(7) Since the outlet tunnel will be used only rarely after the project is completed, the control gate should remain open with closure provided at the intake. A bypass valve and pipe should be provided between the power penstock and the outlet conduit to permit removal of the outlet upstream bulkhead or the emergency gate if the latter is retained.

2. Pursuant to paragraph 6c, EM 1110-2-1150, it is expected that all design memorandums listed in the schedule of design memorandums will be forwarded to your office for approval.

FOR THE DIVISION ENGINEER:

Andrew V. Inge

ANDREW V. INGE
Colonel, Corps of Engineers
Deputy Division Engineer

3 Incl
1. wd 8 cys
Added 2 incl
2. Cy ltr 7 Jun 65, NPDEN-TE, w/1st Ind thereto
3. Cy ltr 28 Oct 65, NPAEN-PR

I

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ENGCW-EZ (13 Nov 65) 2nd Ind
SUBJECT: Snettisham Project, Alaska; Design Memorandum No. 7, General Design Memorandum

DA, CofEngrs, Washington, D. C., 20315, 24 March 1966

TO: Division Engineer, North Pacific Division

Approved, subject to the comments of the Division Engineer in the first indorsement and to the following comments:

a. Section 5. In view of the limited cross section width of the foundation ridge, the high earthquake potential and the low safety factors (1.69-2.43) shown in Figures 15A1 and 15A2, there is a reluctance to support further consideration of Plan A. The waviness of the weak joints is only indicated in a very general fashion in the design memorandum data and its amplitude is not specified. Figure 10A indicates that the maximum slope between nodes must range between 55 and 70 degrees to provide a safety factor of 4.0 at a weak joint with 15 degrees dip which is filled with sandy and silty materials. The potential for downstream movement induced by vertical earthquake vibrations is a further adverse consideration. There is general concurrence with the continued investigation of Plans B and C to provide a firm basis for design. Consideration should be given to a revised alignment for Plan C which moves the north half upstream about 25 feet from the left abutment to the neighborhood of DH-30 and angles downstream across Section L to a point about 50 feet east of the end of the right abutment shown on Plate 8. This alignment would move the dam upstream on Sections H, I, J, and K on Plates 13 and 14. In general, the rock contact at the bottom of concrete should be sloped approximately 10 degrees to cross the planes of potential weak joints. Foundation investigations should be adequate to provide assurance that the base of structure is below all significant weak joints. Similarly the prestressed anchorage of Plan B should be of adequate depth to engage all significant weak joints. The final cost comparison of the two plans should contain all relevant elements, such as special drainage provisions and foundation shaping between high and low areas at the island and channel locations.

b. Paragraph 6.12 and 1st indorsement paragraph 1 d.(2). The use of prestressed tendons to reduce the concrete section of the dam in Plan B does not appear inconsistent with the general safety criteria of the more critical foundation sections below the rock contact and should be considered in the analysis.

c. Paragraph 7.08. The cited rule-of-thumb for rock cover necessary to resist internal tunnel head is unduly conservative. This was probably meant to state a rock cover in feet equal to 3 times the internal pressure in pounds per square inch. In either event, in this instance, a rock cover in feet of 1 to 1½ times the maximum internal tunnel head in feet, subject to the quality of the rock is believed ample for resisting the tunnel head.

d. Paragraphs 7.10 and 8.14. The guide specification for penstock steel is being revised to ASTM-A516, Grade 60, Firebox quality. The new specification is equivalent to A201 modified to include fine grain practice, which is considered worth the small additional cost. This steel should be specified when the thickness of plate does not exceed 1¼ inches. In view of high transportation costs for extra weight, consideration may be given to higher strength steels for economy. Allowable stresses should not exceed 25 percent of ultimate strength. Where rock restraint is relied upon to limit penstock stresses to the normal working value of one-quarter of the ultimate strength, a minimum penstock thickness should be provided to limit steel stress to 80% of yield or 50% of the ultimate, whichever is smaller, under the assumption of no rock restraint.

e. Paragraph 14.03. It appears from the limited data presented in Appendix C, Part 1, that plus 3-inch material is practically non-existent in source A. Since it is feasible to use 6-inch maximum size aggregate in concrete for the dam and 6-inch maximum size can be produced from sources B and C, further studies should consider differences in costs of cementing materials, as well as differences in temperature development within the concrete which would result if 3-inch maximum size were considered in lieu of 6-inch maximum size.

f. Paragraph 14.05. It is assumed that detailed information will be presented on the deposit of apparently suitable sand that exists "within a few hundred yards of source B proper."

g. Paragraph 14.06. This paragraph states that the testing program will include thermal studies on coarse aggregates for future mass concrete temperature studies. It implies that thermal tests will be conducted involving coarse aggregate from sources A and B and reliance will be placed on Dworshak Dam studies for source C. Sufficient testing of source C materials should be performed to establish that the Dworshak data may be used with confidence. It appears that only limited temperature studies will be required due to the relatively small size of structure.

h. Paragraph 14.07. It is doubtful that it would be economically feasible to utilize two sources of aggregates for this project. The preliminary proposal for use of materials from source C will involve approximately 70% of the total project volume of concrete. Therefore, only 30% of the total project volume of concrete would incorporate materials from source B as tentatively proposed. In further consideration of usage, estimates of costs should be based on quantities to be produced from each source if it appears feasible to utilize more than one source of material.

i. With respect to future submission of data, it is requested that a feature design memorandum on concrete aggregate be submitted to include all information on aggregate investigations and test results.

j. Figure 20, paragraph 1 jj of 1st indorsement and EM 1110-2-1150. It is noted that this design memorandum contains no specific sections devoted to hydraulic design information. These studies are necessary to insure satisfactory sizing and operation of many of the project features such as tunnels, surge tanks, penstocks, valves, gates, and for determination of generator and turbine characteristics. If approval of this information is delayed until the feature design memorandum stage, the already tight time schedule shown in Figure 20 becomes impossible. Attention is here invited specifically to the scheduled submittal of design memorandums 13 and 16, and the preparation of plans and specifications based on them. Preparation and approval times should be more realistic. While the typical outline for general design memorandum shown in paragraph 9 of EM 1110-2-1150 does not contain a specific item devoted to hydraulic design, subparagraph b of paragraph 7 states "The various design memorandums should be scheduled and submitted at appropriate intervals in a manner which will meet the indicated purpose of the definite project studies and facilitate orderly design and construction of the project" and paragraph 9 indicates that "any additions required for the specific case at hand" should be included in the general design memorandum. In view of this, unless sufficient time can be allowed between approval of feature design memorandums and preparation of plans and specifications, hydraulic design information sufficient for review and approval of the proposed plan should be provided in the general design memorandum phase.

k. Plate 27. Consideration should be given to using control gates at the intake of the flood control tunnel instead of the flow-jet gate at the end.

l. Appendix C. Mixer grinding studies should be made as suggested on page 4 of the petrographic report.

m. Expanding the digital computer program on Hydro-Power Plant Transients prepared by MIT under contract No. DA-25-075-CIVENG-59-1 with the Missouri River Division to suit the Snettisham Project requirements would furnish a very necessary design tool. The necessary modifications can be accomplished by the Missouri River Division under a contract with Mr. Frank Perkins, the author of the completed MIT computer program. Direct contact with the Missouri River Division to accomplish the necessary work is authorized.

n. 1st Indorsement paragraph 1n. Further planning and design should be based on the conventional surface powerhouse.

o. Although references are made to use of costs of a comparably financed alternative for scoping the project, no costs figures for the

alternative are included in the economic analysis or elsewhere in the memorandum. These costs, the derivation thereof, and the comparability ratio should be included in the economic analysis. In the development of these data consideration should be given to the greater reliability of the at-site thermal alternative with the view to making the comparison on more comparable terms.

p. The comparability test is, basically, an efficiency test to determine whether benefits can be provided by means of a Federal project at less cost in terms of economic resources (land, labor and materials) than by comparably financed alternative means. In the instant case it is noted that 99% of required project lands are currently government owned, and, therefore, no costs for these lands are included in the project costs. To meet the objective of the comparability test an economic value for these lands should be included in the economic analyses. It is noted that contiguous tracts are valued at $500 per acre, which, if applied to the government lands, would increase the project economic costs by about $7,500,000. Although no comparability test results were included in the design memorandum computations, it appears that the project would still be economically justified. In future analyses, a value should be determined for these lands for purposes of economic evaluation. However, the financial analyses should continue to exclude this value.

q. References to division of responsibility for the construction, operation and maintenance of the project should be consistent in citing the authorizing Act as the authority for the division.

r. There is some question as to whether or not the energy and peak power requirements shown in Table 3 and Exhibit 4 are total power requirements of the area or are limited to the requirements of potential power purchasers. Usual procedures call for demonstrating the need for project power based on the total power requirements of the area. Clarification is requested and, if necessary, power requirement data should be revised accordingly.

s. The design memorandum states that, since existing productive capability would not be competitive with the cost of project power, the entire load of the area would be supplied by the project power until fully utilized. This implies that the existing capabilities, indicated to be about 15,000 kw, will be standby or reserve capacity. The FPC recommends that the minimum reserve in any system be equal in capacity to its largest unit. The indicated reserve, therefore, would be insufficient to meet this requirement and, further, would not be able to supply the estimated load requirement indicated in Table 3. Therefore, unless additions are made to the existing system prior to installation of the first unit, consideration should be given to concurrent installation of units 1 & 2. The load data of Table 3 also indicates that in order to maintain adequate reserves, installation of the third unit may have to be advanced two years.

M

ENGCW-EZ (13 Nov 65) 2nd Ind 24 March 1966
SUBJECT: Snettisham Project, Alaska; Design Memorandum No. 7, General Design Memorandum

t. A definitive statement should be obtained from the Bureau of Reclamation to indicate whether or not project power can be marketed at rates sufficient to recover annual project costs, including interest on investment, within 50 years.

u. 1st Indorsement, paragraph 1e. A freeboard allowance of 1.5 feet on the proposed Long Lake concrete dam was approved in OCE 2nd Indorsement, 8 September 1965, on Alaska District letter, 13 August 1965, subject: "Snettisham Power Project Spillway Design Flood," on the basis of information and recommendations presented by the District and Division Engineers in that chain of correspondence and summarized in paragraph 3.21 of the subject design memorandum. An increase in the freeboard allowance from 1.5 feet to 3.0 feet would afford some increase in safety of the dam, including capability to compensate for possible errors in the spillway design flood estimate, errors in spillway rating curves or other factors. On the basis of information presented, there appears to be little reason for increasing the nominal freeboard allowance of 1.5 feet above the peak surcharge level of 903.5 feet expressly to prevent damages from possible wave action. Surcharge levels near the maximum would prevail for only a few hours and surcharge above normal full pool level of 895 should be infrequent. Unless the Division or District Engineers conclude that a higher dam is needed to assure overall safety, the approved 1.5 freeboard allowance should be retained.

v. 1st Indorsement, Paragraph 1 o. The increase in generator ratings will also increase the cost of the turbines.

FOR THE CHIEF OF ENGINEERS:

Homer B. Willis

Incl w/d

WENDELL E. JOHNSON
Chief, Engineering Division
Civil Works

N

COPY

NPDEN-TE (13 Nov 65) 3rd Ind
SUBJECT: Snettisham Project, Alaska; Design Memorandum No. 7, General Design Memorandum

North Pacific Division, Portland, Oregon, 29 April 1966

TO: District Engineer, Alaska District

1. Forwarded to note approval and for action indicated in the preceding indorsements.

2. Paragraphs o, p, r, s, and t of the 2nd Indorsement have been discussed with OCE. Although these comments were generally covered in Design Memorandum No. 3, some points require further clarification and the general design memorandum should briefly summarize all aspects of the project. The questions raised should be covered by indorsement or letter report which would be incorporated in the subject design memorandum.

3. The following comments pertain to the 2nd Indorsement:

a. Paragraph a. The revisions to Plan C proposed in the 2nd Indorsement should be developed as required to establish a cost estimate of comparable accuracy with other plans being studied. The results should be furnished either in the feature design memorandum for the dam or with your reply to paragraph 2 above. The feature design memorandum for the dam should present a thorough analysis and cost comparison for dams having a prestressed and a non-prestressed foundation. The estimate for rock excavation for the prestressed foundation should provide adequately for removal of loose and open rock.

b. Paragraph o. A paragraph summarizing alternative costs, derivation thereof and comparability ratios should be submitted.

c. Paragraph p. If the project comparability test is favorable when a reasonable cost for lands is included in the analysis, no further reply will be necessary.

d. Paragraph r. Reference in this paragraph should be to Table 3 and Exhibit 4 of Appendix A. The comment is raised because Bureau of Reclamation letter (Exhibit 9, Design Memorandum 3) is not clear whether power requirements are the total for the Juneau area or just for loads served by A/J Industries, Inc. and Alaska Electric Light and Power Co.

e. Paragraph s. Design Memorandum No. 3, Exhibit No. 10, indicates local power supplying entities estimate there will be approximately 20,000 kw available standby capacity in Juneau by the end of

1969. Standby requirements based on present planning would be 23,350 kw by July 1971. It should be ascertained if a firm and realistic estimate equal to or greater than this amount can be attained.

f. Paragraph t. OCE does not consider that the letter from the Bureau of Reclamation of 23 April 1965, (Exhibit No. 15, Design Memorandum No. 3) is sufficiently definite.

g. Paragraph u. This office agrees that 1.5 feet of freeboard is adequate.

h. Paragraph v. OCE has clarified this paragraph by pointing out that increases in turbine cost would be those resulting from increased shaft diameter, bearing size, and incidental increases.

4. Request this office be advised by separate correspondence of the date reply required by the above will be received.

FOR THE DIVISION ENGINEER:

GORDON H. FERNALD, Jr.
Chief, Engineering Division

P

SNETTISHAM PROJECT, ALASKA

Schedule of Design Memorandums

No.	Subject	Date
1	Hydrology	15 October 1964
2	Hydropower Capacity	31 October 1964
3	Selection of Plan of Development (Revised)	22 January 1965 7 May 1965
4	Preliminary Master Plan	22 April 1965
5	Access and Construction Facilities	November 1965
6	Deleted	
7	General Design Memorandum	October 1965
8	Preliminary Design Report on Powerhouse	June 1966
9	Transmission Facilities	December 1966
10	Power Intake Works	May 1966
11	Real Estate	December 1966
12	General Geology	June 1967
13	Main Dam and Spillway	September 1966
14	Permanent Operating Equipment	February 1966
15	Buildings, Grounds and Utilities	October 1966
16	Diversion and Outlet Works	May 1966
17	Tailrace and Powerhouse Area Grading	July 1966

SNETTISHAM PROJECT, ALASKA

PERTINENT DATA

(Based on Recommended Plan of Development in Design Memo #7)

LOCATION:

(Near the mouth of Speel River, 28 airmiles southeast of Juneau, Alaska)

AUTHORIZED:

Flood Control Act of 1962, providing for design and construction by the Corps of Engineers and for operation and maintenance by the Bureau of Reclamation.

PLAN: The Snettisham Project consists of a plan to

Construct a concrete gravity dam at the outlet of Long Lake to raise the lake to elevation 895. and to Drill a 8,175-foot power tunnel and a 1,710-foot penstock from Long Lake to a 40,600 KW powerplant at tidewater. Tunnel intake, trashrack and surge tank are included in the waterways construction. From the switchyard, adjacent to the powerplant, a 47.3-mile, 138,000-volt transmission line will extend to a substation near Juneau. Drill a 6,000-foot power tunnel and a 1,690-foot penstock tunnel from Crater Lake to the powerhouse. Increase the powerhouse capacity to 60,900 KW.

PROJECT FEATURES:

Reservoir - Long Lake

Elevation of existing lake surface, feet	815
Elevation of normal full-pool water surface, feet	895
Elevation at minimum operating level, feet	720
Initial active storage capacity, acre-feet	252,000
Active storage after 50-yr. sedimentation, acre-feet	249,500
Active storage after 100-yr. sedimentation, acre-feet	247,000

Reservoir - Crater Lake

Elevation of normal full-pool water surface, feet	1,022
Elevation at minimum operating level, feet	828
Initial active storage capacity, acre-feet	81,000
Active storage after 50-yr. sedimentation, acre-feet	75,000
Active storage after 100-yr. sedimentation, acre-feet	70,000

Hydrology

	Long Lake	Crater Lake
Drainage area, square miles	30.2	11.4
Annual runoff, average (1916-1963), acre-feet	324,300	147,200
Annual runoff, maximum (1943), acre-feet	421,000	192,000
Annual runoff, minimum (1951), acre-feet	252,000	113,000

Long Lake Dam

Type	Concrete Gravity
Length, feet	800
Height of maximum section, feet	110
Crest of dam elevation, feet	903.5
Spillway crest elevation, feet	895.0
Spillway length, feet	225
Mass concrete volume, cubic yards	65,000

Power Tunnels

	Long Lake	Crater Lake
Length, feet	8,175	6,000
Diameter (lined), feet	9.0	7.0
Capacity, cubic feet per second	1,000	450
Intake invert elevation, feet	700	815

Surge Tanks

	Long Lake	Crater Lake
Diameter, feet	25	25
Top elevation, feet	950	1,050
Tunnel invert elevation, feet	660	795
Height above tunnel invert, feet	290	255

Penstocks

	Long Lake	Crater Lake
Length, feet	1,710	1,690
Tunnel diameter, feet	11.0	10.0
Steel penstock diameter, feet	7.0	6.0

Powerplant

Number units	3
Installed capacity, kilowatts	60,900
Operating head, Crater Lake, feet	828 - 1,022
Operating head, Long Lake, feet	720 - 895
Annual firm output, Kilowatt-hours	331,000,000
Gross Average annual non-firm potential, KW-hours	20,800,000

Switchyard

Capacity, kilovolt-amperes	78,000

Transmission Line

Voltage, volts	138,000
Conductor size, MCM - ACSR	795
Overall length overhead section, miles	44.6
Length of steel tower sections, miles	5.3
Length of wood pole H-frame sections, miles	39.3
Length of submarine cable, miles	2.7

Juneau Substation

Capacity - kilovolt-amperes	80,000

SNETTISHAM PROJECT, ALASKA

FIRST STAGE DEVELOPMENT

DESIGN MEMORANDUM NO. 7

GENERAL DESIGN MEMORANDUM

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SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

DESIGN MEMORANDUM NO. 7

GENERAL DESIGN MEMORANDUM

SECTION I - GENERAL

1.01 Authorization.- Snettisham Project was authorized by the Flood Control Act of 1962, Public Law 87-874, in accordance with the plan set forth in House Document No. 40, 87th Congress, First Session, as modified by the reappraisal report dated November 1961. The authorizing act specified that the project will be constructed by the Secretary of the Army acting through the Chief of Engineers and that it will be operated and maintained by the Secretary of the Interior. Pertinent text of the authorizing act is quoted in Design Memorandum No. 1, Hydrology, Snettisham Project, Alaska. This General Design Memorandum, the seventh in a series covering definite project studies for the Snettisham project, is prepared and submitted in accordance with EM 1110-2-1150.

1.02 Purpose.- This design memorandum outlines the studies that led to the selection of the plan proposed herein including all pertinent features of the dam, power intake works, powerhouse and transmission facilities. It also includes an estimated construction schedule outlining the desirable rate of construction in order to meet the increasing needs for power in the Juneau area. Also included is a discussion of local views and cooperation and the coordination with other Federal agencies.

1.03 Scope.- The Snettisham Project will be constructed in two stages. This General Design Memorandum covers the first stage of development which will include the construction of the Long Lake dam and power intake works; the powerhouse structure and the tailrace facilities; the installation of two turbines, generators, and appurtenant facilities; all switchyard and substation structures and improvements as well as necessary station equipment to provide for two generating units' capacity; all transmission lines; all dormitory, office, shop and storage buildings; and all other general property and equipment necessary for the operation of the project. The second stage of development, consisting of construction of the Crater Lake power intake works and completion of the power installation, will be the subject of another General Design Memorandum at a later date.

1.04 The recommendations contained herein are based on detailed mapping in the project area and on extensive foundation explorations and geological evaluations. Design studies were made of the dam foundations and the stability of the structure. Additional studies were made of the transmission facilities and of the features of the power intake

works and power plant, including consideration of both underground and surface powerhouses. The studies were in sufficient detail to prepare comparative cost estimates and to select the most economical plan.

1.05 A scoping analysis to determine the most economical type and height of dam was contained in Design Memorandum No. 3, Selection of Plan of Development; therefore, this study does not include that particular analysis. Economic studies were made to determine the most favorable type, location and elevation of powerhouse and generating equipment; the size and type of tunnel, surge tank and penstock; and the most favorable route, voltage and conductor size for the transmission line.

1.06 Prior Investigations.- The power potentialities of Crater and Long Lakes were initially investigated by private interests, the first in 1913 with subsequent studies made in 1922 and 1928. Although applications were filed with the U. S. Forest Service and Federal Power Commission,the applicants failed to make beneficial use of the water and these applications lapsed. Reports by Federal Agencies included the Federal Power Commission, the U. S. Forest Service, the Corps of Engineers, the Bureau of Reclamation and U. S. Geological Survey. The Bureau of Reclamation began more detailed studies in 1958 and completed a feasibility report in 1959. This report entitled "Crater-Long Lakes Division, Snettisham Project, Alaska" was published in 1961 as House Document No. 40, 87th Congress, First Session. Shortly thereafter the potential industrial power load upon which the power facility had been based was eliminated by the decision of Georgia Pacific Alaska Corporation to forgo a newsprint mill in the area. The project was reanalyzed, and a reappraisal report was completed by the Bureau of Reclamation in 1961. The initial House Document No. 40, as modified by the reappraisal report, provided the basis for project authorization.

1.07 Local Cooperation.- The authorizing act does not require local cooperation for the project.

1.08 Views of Local Interests.- The State of Alaska has expressed approval of the project and has urged the early construction. The proposed project has been explained to and discussed with local groups on numerous occasions, and has been unanimously approved by those in attendance. Officials of the City of Juneau and the local power marketing agencies have expressed a keen and favorable interest in the project. City officials are faced with the prospect of adding additional diesel generation to the present system within the next year, and further additional generation will be required by 1970. For this reason they are extremely anxious that this lower cost hydroelectric power be available to avoid installation of the second additional unit.

1.09 Project Location.- As shown on Plate 1, the Snettisham project site is located in southeastern Alaska near the mouth of Speel River in the Tongass National Forest. It is 28 airline miles southeast of Juneau

and 45 miles from Juneau by water. The project and all immediate surrounding areas lie in a rugged and almost completely unpopulated region with no existing roads and is accessible only by boat or float plane.

SECTION 2 - PROJECT PLAN

2.01 General.- The principal features of the project plan are described and are compared with the Project Document plan in this section. The recommended project plan, as described hereinafter, is shown on Plate 3. Certain modifications of the Project Document plan are recommended. These changes have resulted partially from revised operational planning by the Bureau of Reclamation, who will operate and maintain the project. Additional changes have been made as the result of detailed planning studies and more specific field information.

2.02 Long Lake Dam.- Construction of a dam at the outlet of Long Lake to raise the existing level of the lake by approximately 80 feet to elevation 895 was recommended in Design Memorandum No. 3. Details of the dam are shown on Plate 25. The dam will be a gravity type concrete structure having an overall length of 800 feet and a maximum height of 110 feet. The spillway will be in two sections totaling 225 feet in length, with a crest elevation of 895. The 905 top elevation of the non-overflow sections will provide 1.5 feet of freeboard above the maximum flood pool elevation of 903.5. The outlet works will consist of a 9-foot diameter concrete-lined tunnel. The outlet control gate will be located at the downstream portal of the tunnel. It will be used initially to control the flow through the diversion tunnel during the draining of the lake.

2.03 Reservoir.- Preliminary area-storage curves for the Long Lake reservoir are shown in Figure 1. After the lake has been drawn down during construction of the dam, more accurate surveys will be made and final area-storage curves prepared. The reservoir at full pool elevation 895 will have a surface area of about 1,960 acres. The initial active storage capacity will be about 252,000 acre-feet. No clearing is planned in the area to be flooded by construction of the dam. Further coordination with the Bureau of Reclamation and the Forest Service may, however, indicate the desirability of selective clearing to reduce the amount of debris to be collected at the dam during operation.

2.04 Power Intake Works.- The intake structure will include the trashrack and entrance transitions for both the outlet and power tunnels. The 10 by 27 foot reinforced concrete shaft which will house the bulkhead and gate slots will be approximately 215 feet in height. The operating deck and hoist enclosure at the top of this shaft will be connected to the top of the dam by a 40-foot long bridge.

2.05 The nine-foot diameter concrete-lined power tunnel will extend approximately 8,175 feet from the intake structure at Long Lake to the surge tank. The surge tank will consist of a 25-foot

diameter concrete-lined shaft with a height of 250 feet. A 9-foot diameter steel-lined riser will connect the bottom of the surge tank to the penstock. The butterfly valve, which will serve as the penstock emergency gate, will be located immediately downstream from the surge tank. This valve will be installed in a vault, which will also allow access to the tunnel and penstock. Access to the valve and access vault will be provided by a 350-foot unlined adit.

2.06 A section of 9-foot diameter steel-lined penstock will begin at a point 25 feet upstream from the surge tank riser. This will be reduced to 7 feet through the butterfly valve and the compound bend at the upper end of the inclined section. Approximately 60 feet from the powerhouse, a wye branch will be installed with two 4-foot diameter penstocks continuing to the powerhouse. A short section of the Crater Lake penstock tunnel will be driven, and a portion of the steel lining installed at the time of initial construction. High-strength steel will be used in the lining of the lower portions of the penstocks.

2.07 Powerplant.- The powerhouse will be a conventional reinforced concrete structure located at sea level. The initial installation will consist of two 20,300 KW generating units supplied by the Long Lake penstock. Provisions will be made for future installation of a third unit of equal size to be supplied from Crater Lake. An additional valved connection between the Crater Lake penstock and the second Long Lake unit will be included, as requested by the Bureau of Reclamation. This will allow continued operation of two units from either lake during a shutdown of either tunnel or penstock. The minimum tailwater elevation will be maintained at minus 3.0 by a control sill. An excavated channel will convey the water from the powerplant at low tides. It will have a bottom width of 80 feet and a slope of one foot per thousand.

2.08 Transmission Facilities.- A 138,000-volt, single-circuit transmission line will carry project power to the load center at Juneau. Total line length will be 47.3 miles. The nature of the topography of the route dictates three types of construction. Steel or aluminum tower structures will be used for a distance of 5.3 miles near the Snettisham end. Wood-pole H-frame structures will be used over the remaining 39.3 miles of the overhead portion of the line. Taku Inlet will be crossed by submarine cable using four, single-conductor, 138,000-volt cables. The fourth cable is included as a spare and will be used in parallel with one of the other cables under normal operating conditions. The total length of the crossing will be 2.7 miles. Transformers and switchgear at both terminals of the transmission line will be installed as required in coordination with the installation of generating units in the powerplant.

2.09 Departures from Project Document Plan.- The major departure from the plan presented in the Project Document is the increase in the normal full pool elevation of Long Lake from 815 to 895 by the

construction of a dam. Studies prior to authorization of the Snettisham project indicated the desirability of a dam at the outlet of Long Lake. Although limitations of time and funds for adequate field investigations precluded the recommendation of a dam at that time, it was indicated that further studies of the feasibility of such a dam should be made. Detailed planning studies have therefore been made which resulted in the present recommendation for the construction of the dam.

2.10 Another departure from the Project Document plan is the routing of the transmission line. Planning studies concerning the construction and maintenance problems presented by the previously recommended overland route resulted in the selection of the slightly longer coastal route. Economic studies performed during preparation of this design memorandum also resulted in a change in the recommended transmission voltage from 115-KV to 138-KV.

2.11 Also of significance was the decision by the Bureau of Reclamation to forgo the construction of family housing for employees at the project site. This resulted in the recommendation of a single 20-man dormitory in place of the fifteen family dwellings included in the Project Document plan.

2.12 In addition to the changes in the project resulting from the above-described major departures from the Project Document plan, numerous other changes in the location and characteristics of specific features resulted from detailed planning studies subsequent to project authorization. All of the changes in the Project Document plan are considered within the scope of the Project Document plan and generally have been approved during the course of the pre-construction planning studies.

2.13 Estimated Costs.- The estimated construction cost for the recommended project plan, based on October 1965 price levels, is $53,300,000, excluding interest during construction. Of this cost the first stage development constitutes $40,300,000 and the second stage $13,000,000. These estimated costs are summarized in Table 3 and detailed in Table 5.

SECTION 3 - HYDROLOGY AND WATER CONTROL

3.01 Basin Description.- The area of the Long Lake drainage basin is 30.2 square miles; of this, about 8.4 square miles or 28 percent is covered with permanent snow and ice fields. The elevations range from 814 feet at the existing lake level to 5,193 feet at the highest point. The reservoir is confined between steep rock walls, except at the upper end where a glacial delta slopes gently to the rough terrain beyond. Figure 1 presents the area elevation and storage curves for the Long Lake reservoir. The basin is shown on Plate 2.

3.02 Climatic Data.- The Long Lake drainage lies well within the area of the maritime influence which prevails over the coastal area of southeastern Alaska. Consequently the area has little sunshine, generally moderate temperatures and abundant precipitation. The months of April through July mark the period of lightest precipitation with monthly averages at the lower elevations ranging from 4 to 8 inches. After July, the monthly rainfall increases until the peak month averaging about 20 inches is reached in October. Monthly precipitation averages then tend to decline from October to April. Normal annual runoff from Long River basin and Crater Creek basin is 194 inches and 230 inches, respectively. Normal annual precipitation for these two basins is estimated to exceed these values by 10 percent. Temperature variations, both daily and seasonal, are usually confined to rather narrow limits by the dominant maritime influences. Variations considered on a seasonal basis range from a monthly normal temperature of 25°F in January to 55°F in July. Extremes cover a range of 105°F; from the July maximum of 84°F to the December minimum recorded of -21°F. The average annual temperature is approximately 40°F.

3.03 Snow surveys were initiated at four locations in the Long Lake area during the fall of 1964. The results of one course, which may be considered typical of the powerhouse area, are tabulated below:

Date	3 Mar 65	1 Apr 65	30 Apr 65	5 May 65
Depth, inches	92	73	64	62
Water Content, inches	34	31	29	28

From 1 November 1964 to 30 April 1965, the period during which snow is normally expected to accumulate, 27.95 inches of precipitation was recorded at the Juneau airport. This value compared to the normal of 23.48 inches represents an increase of 19 percent. Assuming the same ratio applied to snowfall at Snettisham, the normal snowpack would be

somewhat lower than the values given above. A more complete description of the climatology of the Snettisham area is included in Design Memorandum No. 1, Hydrology.

3.04 Runoff Characteristics.- Runoff characteristics of streams in southeastern Alaska are subject to maritime influence. This influence greatly increases the runoff per square mile and also changes the timing of high flood flows from that experienced in central or interior Alaska. While flood peaks do occur in May and June, due to snowmelt runoff, the yearly peaks generally center around the month of September. These peaks are due to the intense fall rains. The peak discharge frequency for Long River is shown on Figure 2. Normally, about 75 percent of the annual runoff occurs during the six-month period of May through October. In general, there is very little soil over the underlying rock in the area; hence the facilities for groundwater storage are exceedingly scant and the major components of runoff are mainly surface flow with some subsurface flow and almost no ground water or base flow. Therefore, during dry spells the flow in small surface water streams becomes exceedingly low. The average annual runoff of Long River is 336,900 acre-feet, with 430,500 as maximum and 276,500 as minimum. Average, maximum and minimum annual runoff for Crater Creek are 139,800, 165,000 and 117,000 acre-feet. Maximum and minimum instantaneous peaks for Long River are 5,970 cfs and 2,400 cfs. Crater Creek's maximum and minimum annual peak discharge are 3,100 cfs and 1,270 cfs. The summary hydrograph for Long River is shown on Figure 3.

3.05 Recorded Streamflows.- The U. S. Geological Survey collects streamflow records at Long River and Dorothy Creek, near Juneau. Streamflow records used in this study are as follows:

Station	Period of Record	Drainage Area (sq mi)
Crater Creek at Crater Lake Outlet	Feb '13 to Dec '20	11.4
	Jun '23 to Sep '23	
	Jun '24 to Sep '24	
	Jun '27 to Dec '32	
Dorothy Creek	Oct '29 to Oct '41	15.2
	Sep '42 to Dec '43	
	Jun '44 to present	
Long River	Oct '15 to Sep '24	32.5
	Oct '26 to Dec '26	
	Jun '27 to May '33	
	Oct '51 to present	

3.06 Simulated Monthly Flows.- The present practice in design of water resource projects includes the routing of monthly streamflows based on past records through project facilities in order to determine project accomplishments. A system of synthesizing monthly runoff is used to overcome some of the deficiencies in the present practice of assuming that historical runoff will be repeated. Synthesizing permits a reasonable mathematical determination of expected project benefits insofar as hydrologic factors are concerned. In order to examine the project under more realistic assumptions, forty independent 50-year periods of monthly records were generated synthetically. The procedure followed is outlined in "Technical Bulletin No. 1, Simulation of Monthly Runoff," published by the Hydrologic Engineering Center of the U. S. Army Corps of Engineers, Sacramento, California, in November 1964. The project was then examined using higher than and lower than normal flows for a 50-year period. The results of this examination shows that should the 50 years be wetter than normal, the effect on prime power would be negligible. However, should the 50 years be dryer than normal, the effect would be a reduction of up to 3 percent in the amount of prime power that could be generated. A frequency curve was derived using the values of the developed forty 50-year periods and is shown on Chart 4 of the Power Appendix. As may be seen from this chart, there is a 90 percent chance that the average annual volume in acre-feet for the next 50 years will be in the range of 316,500 to 330,400 acre-feet. The following is a tabulation in descending magnitude of the average flows in acre-feet per year of the forty 50-year periods:

AVERAGE ANNUAL FLOWS
PER 50 YEAR PERIOD
ACRE FEET

333,700	325,600	323,400	320,800
331,800	325,500	322,900	320,800
328,600	325,500	322,700	320,500
327,200	325,300	322,700	320,100
326,900	325,100	321,900	319,200
326,700	324,500	321,800	319,000
326,300	324,400	321,500	318,400
325,900	324,200	321,300	318,000
325,800	323,800	321,300	318,000
325,600	323,500	320,800	311,200

3.07 Spillway Design Flood.- Since a dam will be constructed at the outlet of Long Lake, derivation of a spillway design flood is necessary. Both a spring flood due to rain and snowmelt and a fall

flood due to rain were developed to determine which would be more critical. The IBM 1920 computer in the North Pacific Division office was used in these developments with general guidance provided by their Water Control Branch. Criteria used in the study are as follows:

a. May, three-day probable maximum precipitation of 24.4 inches.

b. September, three-day probable maximum precipitation of 33.2 inches.

c. Snowpack water equivalent of 52 inches on 15 May.

d. Losses to groundwater and atmosphere considered negligible due to the underlying rock, high humidity and preliminary snowmelt.

e. Two routings of the May flood were made with initial lake stages at 875 feet and 895 feet, respectively.

f. September flood routed using an initial pool elevation of 896.3 feet.

g. Routing phases were varied until a reasonable lag time was found.

h. A high base flow of 1,400 cfs was assumed during the September storm to account for glacial melt and anticedent rains.

3.08 Critical hydrometeorological data were supplied by the Hydrometeorological Section of the U. S. Weather Bureau. Solar radiation was calculated using CRREL's Research Report No. 160, "Daily Sums of Global Radiation for Cloudless Skies." Snowmelt rates were calculated using EM 1110-2-1406, "Runoff from Snowmelt." The snowmelt computations are shown on Table 1. The block diagram which depicts the sub-basin divisions used in the computer program is shown on Figure 4. These sub-basins are also shown on the Basin Map, Plate 2.

3.09 Two lake stages, 875 and 895, were used in the May flood routing as it is quite probable that the lake will be drawn down low every spring. Pool regulation and power studies indicate that the maximum May elevation would be 855 feet. Twenty feet was arbitrarily added to this as a safety measure. During the September flood, it is reasonable to assume that the pool will be up to the spillway crest elevation of 895 feet. At the start of this flood, it was also assumed that of the 1400 cfs base flow, 1000 cfs would be passing over the spillway, with 400 cfs passing through the powerhouse. This 1000 cfs makes an initial pool elevation of 896.3 feet.

3.10 The September flood is the most critical with a peak inflow of 30,400 cubic feet per second (cfs), a peak outflow of 21,300 cfs and a maximum pool elevation of 903.5 feet. The May flood, with starting pool elevation of 875, produced a peak inflow of 22,800 cfs, a peak outflow of 14,700 cfs and a maximum pool elevation of 901.8. With starting elevation at 895 and a peak inflow of 22,800 cfs, the peak outflow is 16,100 cfs and the pool reaches a maximum elevation of 902.1 feet. The above values are for a 225 foot wide spillway. The September and May floods are shown on Figures 5 and 6, respectively.

3.11 Spillway crest elevation is 895 feet and the width is 225 feet. A width of 225 feet was originally chosen working with a design head of 10.5 feet. When the floods were developed and routed, widths of 200, 225 and 250 were used. This narrow range in width selection was maintained because of the physical characteristics of the dam site. The results of these routings of the September flood with initial lake stage at 896.3 feet and a spillway crest of 895 feet are as follows:

Spillway Width Feet	Peak Outflow cfs	Elevation Feet
200	20,000	903.8
225	21,300	903.5
250	21,300	902.9

3.12 Ice.- Long Lake normally freezes over in November and is usually free of ice by late June although isolated blocks of ice still exist in late July. Ice measurements made in March of 1965 revealed thicknesses between 4 and 5 feet over the entire lake. The upper foot was composed of layers of snow and ice saturated with water and partially refrozen. It is believed that the saturated upper layer is caused by the depression of the original ice layer by the weight of subsequent snowfall. This allows lake water to overflow onto the original ice surface. This overflow water then freezes and the process is repeated. This ice cover will have no significant effect on project operations due to the depths at which the intakes will be located.

3.13 Sedimentation.- There are no data presently available concerning sediment loads carried by the streams tributary to Long Lake reservoir. The streams are predominately glacial and as such carry suspended sediment and bed loads. Measurements of the suspended sediments in the existing lake were made in the spring and summer of 1965. The results of these data are summarized in Table 2. The first set of observations were made in March 1965 through

5 to 6 feet of ice cover. The second set of observations were made in July 1965 after virtually all of the ice cover had melted from the lake and overturning had occurred. These observations indicated an extremely low concentration of suspended sediments. Observations of the delta formed at the head of the lake indicates that a large amount of bed load material is being deposited. This material is water borne and a fairly rapid rate of deposition is indicated. This material ranges in size from 4 inch cobbles to fine sands. In addition to the material being transported to the lake by water, a substantial amount is introduced by the action of snow slides. Some of this latter material is organic and most of the material is deposited in the portion of the lake that will become inactive storage.

3.14 Because of the lack of information on the actual sediment load carried into Long Lake reservoir, it is not possible to compute the annual storage depletion by direct methods; however, a reasonable estimate may be made by indirect methods. The Tonsina River, located in the Copper River Basin, has a drainage area with characteristics similar to that of Long Lake. Forty-seven suspended sediment measurements made during the last few years indicate an average suspended sediment concentration of 113 parts per million. Applying this ratio to Long Lake's average annual inflow of 324,300 acre-feet yields an annual sediment inflow of 37 acre-feet per year. Measurements of the suspended sediments near the lake outlet indicate an average concentration of about 5 parts per million. This would indicate a suspended sediment outflow of about 2 acre-feet per year. Thus about 35 acre-feet per year are contributed to the reservoir by suspended sediments. No data are available concerning movement as bed load; however, for the purpose of this study an average annual contribution of 15 acre-feet has been assumed, thus yielding an average annual total storage depletion of 50 acre-feet. Observation of the uniformity of the suspended sediment concentration throughout the lake indicates that almost all of the inflowing materials are deposited shortly after reaching the lake waters; therefore, it may be assumed that all of the storage depletion will occur in live storage. It is tentatively planned to establish seven "Detailed Study Ranges." A stream gaging station is proposed at the location shown on Plate 2. Suspended sediment and bed load samples will be collected at this station.

3.15 An interesting phenomenon may be observed when the lake is drawn down for construction. It is planned to draw the lake level down to elevation 680. This is well below the minimum power pool and it is expected that the inflowing waters will wash some of the sediments out of the active storage area and deposit them in the dead storage area. In order to examine this it is planned to map the exposed floor of the reservoir immediately after drawdown and just prior to refilling. A revised storage area curve will be prepared.

3.16 Hydrometeorological Data Instrumentation.- In order to insure efficient utilization of the projects water resources, it is considered necessary to install instruments to measure and telemeter precipitation, temperature, wind, reservoir stage, tailwater and tide stage and the accumulated snowpack in the area tributary to the reservoir. It is presently proposed that the telemetered information will be received and recorded in the powerhouse control room.

3.17 Two permanent climatological stations are proposed: a standard cooperative station in the camp area and a second station to be in the area tributary to the reservoir. Information from the second station will be telemetered to the powerhouse. An anemometer will be installed near Taku Inlet. Data from this installation will be used in the transmission line design and the site will be abandoned when this need is fulfilled.

3.18 A reservoir stage gage will be installed during the construction of the dam and outlet works. Because of the wide range in reservoir elevations it will not be feasible to install a vertical shaft for the float gage. An inclined gage similar to that developed by the U. S. Bureau of Reclamation for operation of their Eklutna project or a bubbler type gage will probably be installed. It will be necessary to transmit this information to the powerhouse either by a control cable or a remote radio system. A standard float tailwater gage will be installed on the downstream side of the powerhouse. This information will also be telemetered to the powerhouse.

3.19 The snowpack in the area tributary to the reservoir will be measured by a Pressure Pillow of the type recently developed by the U. S. Soil Conservation Service. This device has been tested and has worked well on Mt. Hood, Oregon, where the snow conditions are quite similar. This information will be telemetered to the powerhouse. Additional snow courses may be established, if needed.

3.20 The estimated costs for the above instrumentation are as follows:

Climatological Stations	$ 5,000
Anemometer	3,000
Reservoir Stage Recorder	5,000
Tailwater Stage Recorder	3,000
Pressure Pillow and Snow Courses	5,000
Tide Stage Recorder	3,000
Telemetering Network	24,000
Contingencies	7,000
Total Cost	$55,000

3.21 Freeboard Allowance.- The freeboard allowance is 1.5 feet, giving a top of dam elevation of 905 feet. This allowance is based on the following criteria;

a. The pool will be at maximum elevation for only a few hours during the Maximum Probable Flood.

b. The material upon which the dam will be built is rock, not susceptible to eroding, and overtopping of the dam would be non-damaging.

c. The fetch for wave build-up is quite short, with 40 miles per hour winds producing two-foot waves. However, as the Probable Maximum Precipitation storm comes from the sea, the wave producing winds during the storm would send waves to the opposite end of the lake.

d. The downstream channel and vicinity is steep, rocky, unpopulated and has a short distance of five miles to tidewater.

3.22 Tailwater Data.- The powerhouse and operating facilities are not located in the valley below the dam; therefore, releases from the spillway or outlets will not affect these designs. The powerplant housing and operating facilities will be located slightly above sea level in the area indicated on Plate 3. In order to accurately determine the range of tides to be expected in the project area, a recording tide gage was installed on Speel Arm at the point indicated on Plate 2. This gage was placed in operation on 11 January 1965 and tidal records from that time to 24 August 1965 were collected and analyzed. Highest and lowest expected tides were obtained by a direct correlation with the highest and lowest tides observed at Juneau. The results of these studies are presented in the following tabulation:

	Snettisham Local Datum	Juneau MLLW
Highest Expected Tide	11.4	22.6
Mean Higher High Water	4.76	16.40
Mean High Water	3.73	15.40
Half Tide Level	-2.90	8.50
Mean Low Water	-9.52	1.60
Mean Lower Low Water	-11.14	0.00
Lowest Expected Tide	-16.8	-5.8
Tidal Range	28.2	28.4

The tide data described above is based on a seven-month period of record. This is not considered to be enough record for an absolutely accurate determination of the tidal characteristics; therefore, after one Lunar year of record has been obtained, another analysis will be made. It is not expected that these latter values will differ from the previous values by an amount significant enough to affect the project design.

3.23 Design Wave on Speel Arm.- A storm wave of 2.9 feet has been computed for the camp and powerhouse area. This is based on a 60 mile per hour southerly wind. This wind is considered to be sufficiently rare for design standards.

SECTION 4 - GENERAL GEOLOGY AND SITE INVESTIGATIONS

4.01 Background Information.- Although much of the following general geologic background information is similar to that submitted in Design Memorandum No. 3 (January 1965), a number of significant changes or modifications have been developed as a result of further evaluations and studies performed during the recent 1965 field season. This report presents results of the most recent explorations and office studies and, therefore, supersedes the geological information contained in Design Memorandum No. 3.

4.02 Introduction.- Both the geographical and geological settings for the Snettisham hydro-electric development present many important features which will have a significant influence on the planning, design, construction and ultimate operation of the project. The following summarized presentation of this environmental setting is based upon background data contained in United States Geological Survey publications, and upon extensive field investigations and studies conducted by Alaska District during the 1964 and 1965 field seasons.

4.03 Geographical Setting.- The Snettisham project is located within the Tongass National Forest on the Alaska mainland, approximately 28 air miles southeast of Juneau (Plate 1). The project area lies in a very rugged and almost completely unpopulated region with no existing roads, and is accessible only by boat or float plane. The climate, typical of southeastern Alaska, is characterized by moderate temperatures at sea level, mild winters, cool summers and heavy precipitation, mostly in the form of rain. The annual rainfall averages approximately 140 inches at the project and snowfall during winter months sometimes accumulates to depths of ten feet or more. Forest cover with accompanying dense undergrowth extends from sea level to a maximum altitude of approximately 2,500 feet, but these timbered areas are often broken by large muskegs which do not support tree growth.

4.04 Regional Geology.- A complete treatment of all aspects of the regional geology and of its application to the development and operation of the Snettisham project will be submitted in a "General Geology Design Memorandum" scheduled for completion in June 1967. The following discussion provides a brief but overall geological background of southeastern Alaska covering the essential features of the geological environment commensurate with the scope of this report.

4.05 The southeastern coast of Alaska is generally a coastline of submergence resulting at least partially from geologically recent rises in sea level. As such, it has well developed drowned river valleys or "fiord" type features wherever westward flowing rivers meet

the sea (Plate 4). These "fiord" type features have also been caused by the unusually deep scouring of river valleys in their lower reaches by past glacial action. Extensive stream aggradation with resultant valley clogging by alluvial sediments has occurred in many of the larger river systems near their mouths. When combined with the unusually high tidal ranges in the area, this has given rise to extensive tidal mud flats along portions of the inundated coastline. As an overall result, many streams have braided and meandering patterns throughout the lower reaches of their drowned valleys. In sharp contrast to these gentle stream gradients near the immediate sea coast, many precipitous stretches occur as streams emerge and cascade down from steep mountain fronts which parallel the irregular coastline. Most major streams have active glaciers at their headwaters and their courses from these points of origin to the sea are usually well marked by typical glacial erosional features such as "U"-shaped valleys, cirque lakes, hanging tributary valleys, truncated spurs, and morainal deposits.

4.06 The country rock of this portion of southeastern Alaska consists of rock units derived from and associated with the Coast Range batholith. This batholith is a very extensive and complex body of igneous and metamorphic rocks which generally parallels the Pacific coastline and trends, therefore, in a roughly northwest-southeast direction (Plate 4). The intrusion of the batholith into formerly existing sedimentary rocks is believed to have occurred in Cretaceous time (approximately 120 million years ago) probably during the "Laramide Revolution." This represents an epoch of geologic time in which many such world-wide batholithic intrusions and mountain making processes were taking place on earth. The intruded area has since been re-uplifted by orogenic movements to form the present Coast Range of mountains, and as such now contains igneous rocks of the original batholith in addition to composite mixtures of igneous and metamorphic rocks derived from alteration of the pre-Cretaceous sedimentary rocks.

4.07 This entire uplifted igneous-metamorphic complex has since been highly modified by the various agents of erosion. Most prominent of such agents has been ice from all four of the major continental ice sheets of the Pleistocene epoch and from associated alpine glaciation. Water erosion by rivers and streams has also played an important role in sculpturing the mountainous landscape of this general region, but most of such subsidiary erosional patterns are superimposed on the more dominant glacially carved features. To a large extent the most prominent erosional features have been developed along joints, faults and lithologic boundaries, which in past times have afforded initial degrees of differential erosion along which water courses and later glacial courses could become firmly established. Most major valleys and even minor tributary draws throughout this area are therefore topographic expressions of the primary and secondary rock structures of the region. Major faulting and much of the jointing reflected by this topographic expression has resulted from crustal movements which have taken place after the intrusion

and consolidation of the Coast Range batholith. These crustal movements are believed to have taken place in conjunction with the latest orogenic upheaval resulting in the formation of the present Coast Range mountains, but some such rock cleavage may also be related to earlier stresses resulting from cooling of the original batholith.

4.08 Isolated zones of heavily fractured or decomposed bedrock that are now seen to exist are almost always associated with faulting or with close systems of jointing. With the exception of these localized zones of rock deterioration, few deep pockets or area-wide zones of deep residual weathering are believed to exist throughout the extensive reaches of the Coast Range mountains. This is believed to be due primarily to the very recent advance and retreat of regional glaciation in the area. Such a noticeable lack of widespread, deeply weathered zones in bedrock in an area of 142 inches of annual rainfall is a very rare phenomenon and can only be explained by the efficient removal of nearly all regolithic materials by glacial scour. This scouring has been sufficiently recent to prevent a build-up of significant further residual weathering products on the bedrock surfaces. Accumulations of unconsolidated materials derived from these sources and transported from their points of origin may now be seen in debris cones at the heads of glacial lakes, in alluvial fan outwash deposits and as alluvial or moraine fillings in the canyons and valleys of the region.

4.09 Project Site Geology.- The predominant rocks which occur within the immediate area of the Snettisham project consist of quartz diorite, gneiss, and some localized phases of biotite schist which all occur in a somewhat interwoven and random distribution pattern throughout the main rock body at the site (Photo 24). Due to recent glacial scour in this area and resultant removal of most weathered surface rock materials, the majority of the bedrock which will be encountered by proposed engineering structures will be relatively fresh, dense and of durable quality. Localized exceptions to this will definitely occur, chiefly in association with the several major shear zones which occur in the area. These relatively narrow zones in which the rock has been crushed and broken have allowed much easier access to percolating ground waters and these waters have imposed a substantially higher degree of chemical alteration on the sheared rock than is to be found in other non-disturbed portions of the general rock body. Granitic type rocks such as quartz diorite and gneiss contain high percentages of the feldspar mineral plagioclase which upon chemical decomposition by ground waters with dissolved carbon dioxide (carbonic acid) yield the clay mineral kaolin. In connection with this feature, there has been found to exist in surface exposures from one to six feet of plastic fault gouge, mylonite and associated lesser quality rock materials along the central portions of the larger shear zones. Borings which have penetrated these same fault zones at depth have shown less clay gouge than at surface exposures (Photo 20). This is

probably due to the greater amount of weathering and kaolinization which occurs at the ground surface than at depth. Adjacent to and occurring in relatively narrow belts parallel to the central clay gouge seams are broken and weathered zones of less competent rock, which represent border phase fracturing within the stress influence of a given shear zone. Because of the overall lesser quality rock materials associated with the larger shear zones and to some extent the close systems of joints, exact locations and orientations of all such pertinent rock structural features with respect to the locations of proposed engineering structures is a factor of considerable design importance. (See geologic map, Plate 6.)

4.10 Seismic Considerations.- While the presence of major fault zones does testify to the existence of seismic activity in this immediate area during the geologic past, it is generally believed that all such movements along these faults are sufficiently remote in geologic time as to preclude concern of their further adjustments during the life of the project. To the extent possible, attempts to confirm this premise have been made by searching for obvious offsets in recent glacial or alluvial strata at points where they cross known major faults, but thus far no such evidence of recent faulting activity has been found anywhere near the immediate project area. The proximity of the Snettisham Project to a major zone of crustal weakness does, however, make it prudent to design important structures at the project for fairly substantial earthquake accelerations. Part of this major crustal zone of weakness is called the "Pacific Ring of Fire" and is a relatively narrow world-wide belt along which some 80 percent of the world's major earthquakes occur in addition to appreciable amounts of volcanic activity (Figure 7). The Pacific belt starts below the southern tip of South America, extends northward along the entire southeastern and southern coast of Alaska and continues along the Aleutian Chain finally curving to the southwest toward Asia. The 1957 seismic probability map by the Coast and Geodetic Survey has placed the area in which the Snettisham Project lies in seismic Zone I (Plate 5). Inasmuch as these very arbitrary boundaries are very compressed in this area and earthquake nagnitudes of 8 plus have been recorded within only 80 miles of the project, the Snettisham Project is considered as being in Zone III for which magnitudes of 6.0 and above would be anticipated. Use of Zone III magnitude was recommended in Design Memorandum No. 3 and approved by higher authority. This means that the design of all major rigid structures such as a dam will include horizontal earthquake accelerations of 0.1 gravity or greater, and other more flexible works such as building structures will assume minimum "Z" values of 1.0 (EM 1110-345-150).

4.11 Except for the Long Lake tunnel, no important structures will be constructed across major fault zones. This tunnel will pass through what are considered to be three major faults (Plates 6 and 20). A dam at Long Lake outlet would have several minor faults in portions of its

foundation (Plate 9). In all cases of such required association of engineering structures with known fault systems, remedial treatment will generally be directed toward leakage control and replacement of lesser quality rock materials with dental concrete. No special design is planned to compensate for any movement along such faults within the life of the structures.

4.12 Geologic Explorations.- During the 1964 field season 22 holes were drilled (6,678 linear feet), 37 hand auger borings and 68 test pits were dug. During the 1965 field season 40 holes were drilled (2,884 linear feet), 36 auger borings and 58 test pits were dug. In addition 6,000 feet of seismic line were run in the 1965 field season.

4.13 Plates 6 and 6A show the locations and types of all explorations performed to date at the site. Sixty-one core drill holes, comprising a total of 9,512 feet of NX and BX diamond drilling and 50 feet of 6" drilling have been completed to date by Alaska District on all features for the project site investigation. Of this total, 34 holes (4,309 feet) have been drilled at the Long Lake dam site (Plates 7 and 8 and Logs in Appendix C); three holes (552 feet) have been drilled at the Long Lake intake structure locations; three holes (1,613 feet) have been drilled along the power tunnel alignment; two holes (1,295 feet) along the penstock alignment; and 20 holes (1,700 feet) in the underground and surface powerhouse areas (Plates 17, 18, 19 and Appendix C). In addition to these exploratory borings, the Bureau of Reclamation had previously drilled three holes (104 feet) in the Long Lake dam foundation and two holes (92 feet) in the surface powerhouse area. Explorations conducted for the purpose of natural concrete aggregate investigations to date consist of shallow test pits and five drill holes (Appendix C). Foundation investigations along possible access routes and at the dock site consist of 61 hand auger borings, eight vane shear borings, and 34 wash probe holes.

4.14 Other Types of Investigation.- All of the foregoing subsurface exploratory work has been augmented by geologic mapping, seismic surveys, bore hole photography, underwater TV camera study (Photo 11), SCUBA diver-geologist examination (Photo 10) and laboratory testing of materials obtained from drilling and test pitting operations. Results of these other investigation studies are included wherever appropriate along with the general treatment for each of the project features discussed in this design memorandum. A complete report on the lake bottom diving reconnaissance as written by the SCUBA diver-geologists from Scripps Institute is included in Appendix C.

SECTION 5 - GEOLOGIC EVALUATION OF DAM SITE

5.01 Geologic Nature of Dam Site.- Power and economic studies have shown that construction of a dam at the outlet of Long Lake to raise the pool level to elevation 895 will enhance the overall economic feasibility of the project. The relatively restricted level area available at the outlet of Long Lake, combined with several adverse effects of the valley geometry, has sharply limited the choice of dam designs which are economically feasible at this site. Foremost among these adverse features are the fast drop-offs in rock slopes which occur both upstream and downstream of the site and which continue for several hundred feet below the highest point of the lake outlet ridge where any dam axis would have to be located (Photos No. 2 and 5). This rather narrow rock ridge remnant now remaining at the lake outlet after the last glacial retreat has caused a serious problem in obtaining adequate base widths for proposed dam structures.

5.02 One of the first features of this unique dam site that required investigation was the true nature of the rock mass, the topmost portions of which now form the present dam site ridge. The chance that this rock mass might have been a landslide rather than a resistant ridge of true bedrock was a possibility that warranted close investigation. However, subsequent geologic and foundation studies show the rock mass at the outlet to Long Lake has an observed and proven continuity of structural geology and lithology across the dam site and well into the abutments. In addition to this principal method of proof, the following other lines of evidence are believed to definitely substantiate the premise that Long Lake dam site is not a rock slide mass: (1) there are no visible scars on either mountainside where such a large slide might have originated; (2) in-place bedrock can be followed by eye along outcrops from the dam site ridge to points several hundred feet lower on the downstream side; (3) pressure testing of bore holes shows generally tighter rock with depth; and (4) SCUBA diver-geologists made careful note of glacial grooving and striations along the underwater upstream face of the dam site ridge. (See Photo 10 and Diving Report, Appendix C.) Only in-place rock could have resisted and deflected the advancing glacier upward at this point.

5.03 Suitable Dam Types.- Because of the narrow and steep dam site ridge, both earth and rockfill type dams with their relatively large base width to height requirements have been eliminated from consideration as feasible developments at this site. Some preliminary studies were made on the possibility of constructing a rockfill dam, chiefly because of the unlimited quantities of excellent quartz diorite bedrock available at the site for the manufacture of rock shell materials. Even with a low estimated unit price per cubic yard for rock, the engineering problems connected with such a design at this site would make it of doubtful physical and economic feasibility. Important among these engineering design problems would be the required construction of concrete retaining walls on steep rock surfaces to catch "splinter fills" from rockfill

slopes, and the thick accumulations of silt and debris which are known to comprise the foundation conditions at the bottom of Long Lake. The foregoing problems are most severe for dam design heights some 200 feet above present Long Lake level, but remain to a sufficiently important degree for all other lesser dam heights studied; therefore, no further consideration has been given to rockfill or earthfill type dam designs in this memorandum.

5.04 Long Lake outlet was also studied as a possible site for a thin arch or gravity-arch type dam. Unfavorable valley geometry, encountered at a prominent topographic break in the rock slope on the left abutment (See Plate 7 and Photo 2), would require thrust block construction. The curved axis for any arch dam would also have to be shifted substantially upstream into Long Lake in order to assure a sufficiently massive left abutment of rock which would be capable of resisting the arch thrust (Photos 4 and 7). This required upstream shifting of any proposed arch dam axis would place major central portions of the arch structure in Long Lake where adequate bedrock foundation has been shown by barge drilling to lie at infeasible depths (Photos 12 and 14).

5.05 With virtual elimination of earth, rockfill, thin arch and gravity-arch type dam designs for the foregoing reason, all efforts were directed toward concrete dams with minimum base width-to-height requirements. The most obvious design form the combined standpoints of simplicity, conservatism, and minimum likelihood of hidden costs arising from latent foundation conditions is the concrete gravity dam. Much of the field and design effort in this General Design Memorandum has been directed toward proving up the foundation and material requirements for a concrete gravity dam at Long Lake outlet which would raise the lake to elevation 895.

5.06 Gravity Dam Foundations.- Investigations and studies of the dam site ridge for a concrete gravity dam foundation have been carried on for two field seasons. The results of these investigations have tended to demonstrate that the quartz diorite bedrock is highly competent from the standpoint of almost any vertical loading which might be imparted from a proposed dam structure. The principal problem that has emerged from these studies is the question of dam site stability with respect to the horizontal loading forces imparted from the reservoir and by potential earthquake accelerations. There are two important aspects of this dam site ridge which, when considered together, serve to reduce its inherent resistance to these slide inducing forces: (1) the heretofore mentioned ridge character of the dam site which results in lack of downstream restraint or toe support for the upstream portions of the dam foundation (this type of support can usually be taken for granted in the average dam site) and (2) the proven existence of low angled to a slightly downstream dipping joints which "daylight" in both upstream and downstream areas of the dam site ridge, and which are potential planes of weakness within the foundation (See Photos 7 and 9 and Plates 11 through 14).

5.07 A great deal of investigative effort has therefore been expended to ascertain the true nature of all major low angled joint surfaces that exist within the zones of foundation influence of the proposed gravity dam. The results of these investigations may be briefly summarized as follows: (1) the overall structural attitude of the more important low angled joints varies from a 3 degree dip upstream (island area) to a 15± degree dip downstream (south of island area); (2) the persistence with depth of the more continuous and seemingly major low angled joints appears to end below elevation 785 ± . This would tend to lend confirmation to the speculative origin of many such joints as being associated with an unloading type phenomenon related to the relatively recent withdrawal of several hundred feet of glacial ice load; (3) drill water loss often encountered at low angled joints demonstrates that these joints are open at least in some areas. Surface exposures of these joints (Photos 7 and 8) indicate rather tight openings but with a thin zone of decomposed granitic rock which is altered to a sand or "grus." In a few drill holes, the joints in question would appear to have relatively good interlock with associated fresh rock immediately adjacent to the joint surface. The soft unconsolidated clay-type material encountered on low-angled joints in two drill holes is considered to be a purely local condition not important to the foundation, since additional borings to define this condition disclosed nothing but clean joints at the same elevation.

5.08 Explorations and observations at the dam site show strong evidence that the low angle joints are continuous or so nearly continuous that hard rock cohesion across the joint cannot be assumed. Observed undulations in the hard rock structures along the joints have not attained angles anywhere close to 90 degrees, which would be a further requirement to insure shear resistance. Openness and continuity of joints is further demonstrated by two instances where pressure tests caused cuttings to be washed into holes being photographed 20 and 26 feet distant (Photo 25). It must be assumed, therefore, that the low angled joints are generally continuous, lack appreciable cohesion, and that the overall average shearing resistance along any major low angle joint surface will be of a relatively low order of magnitude. (See Figure 9 and associated notes.)

5.09 Because these major low angled joint surfaces are suspected as being potential planes of weakness, two additional "sliding planes" have been analysed in stability analyses. (Figures 8, 9, 10, 10A and 15A). In the island area of the dam site the major sliding plane as mentioned in paragraph 5.07 is found at Elevation 785 and is approximately horizontal over the central one third of the site (Area I). In the area of the dam site south of the island (also approximately one third of site), the major sliding plane dips downstream at an approximate 15 degree angle (Area II). These two design sliding stability planes within the foundation of the dam site are in addition to the standard planes analyzed through the concrete of the dam and at the rock-concrete

contact at the base of the dam section. As described in the following paragraphs and demonstrated on stability analysis plate 15A the two potential sliding planes in the foundation offer the least net design resistance to sliding and are, therefore, considered the most critical ones to study.

5.10 <u>Three Alternate Plans for Foundation Treatment</u>.- In general recognition of the potential problem of dam site sliding stability, three basic plans with accompanying cost estimates are presented in this Design Memorandum. Plan A (See Plate 24) would build a concrete gravity dam along the highest level areas of the dam site ridge with a minimum of rock excavation. No special foundation treatment would be performed other than the standard curtain grouting, drainage, and surficial foundation preparation. A design safety factor against sliding equal to at least 4.0 would be obtained only by taking sufficient tangible credit for shear resistance along the appropriate low angled joints. This maximum shear requirement approximates 60 pounds per square inch without earthquake consideration. Plan B, the recommended plan, also involved a minimum of rock excavation but takes considerable lesser amounts of design credit for shear along a given low angled joint. Plan B develops sufficient additional frictional resistance on these joints by foundation pre-stressing to obtain a safety factor against sliding equal to 4.0 (without earthquake). The pre-stressing would be accomplished over the more critical central two thirds of the dam site (Figures 9 and 10 and Plate 25). Plan C would remove rock down through the lowest major low angled joint (Elevation 785 $\pm$) across the central two thirds of the dam site and result in a substantial increase in mass concrete requirements for the dam structure (Plate 26).

5.11 The order of increasing conservatism in foundation design treatment varies approximately in the order, Plan A, Plan B and Plan C. As might be expected, the cost for foundation treatment increases in the same order (Table 6). Although Plan A has the least cost and requires relatively small shear values for design adequacy, studies clearly demonstrate that even the nominal shear value of 60 $\pm$ psi cannot be relied upon. Furthermore, such studies have been conducted to a point that it is very doubtful if additional explorations could demonstrate the existence of that strength. Without such conclusive evidence, Plan A cannot be recommended. Plan C is an admittedly conservative approach that would probably insure a sufficient safety factor against sliding, but at a higher cost. Plan C would also remove a substantial load on the foundation during required rock excavation and this "unloading" could result in new openings and extension of existing minor low angled joints deeper into the foundation. This risk could not occur in Plan A, nor particularly in Plan B. Therefore, Plan B might be recommended over Plan C even if there were no appreciable cost difference. Plan B, the recommended plan, would provide the essential elements of both safety and economy in one overall scheme.

5.12 Foundation Treatment - Plan B.- When only nominal design credit is taken for shear along a low angled joint surface, the greater amount of design resistance to movement must be developed from frictional forces. It has been determined from stability analyses that the natural frictional force alone is insufficient to yield a 4.0 safety factor against sliding. The two principal factors which determine the frictional force which can be developed between any two surfaces are the coefficient of static friction and the total load applied normal to the surfaces. The most favorable method of increasing this normal loading force is by pre-stressing the foundation with high tension cable strands. The foundation pre-stressing plan shown for Plan B on Plate 25 and Figures 9 and 10 has tailored the amount of pre-stressing required in a given area of the dam foundation to yield a safety factor against sliding of 4.0 (without earthquake).

5.13 Recognizing that reduced shearing strengths do exist along most of these joint surfaces and in order to introduce reasonable economies into the pre-stressing plan, it has been deemed advisable to assign minimal shear strength values to the major low angled joint surfaces. The allowable design shear values considered reasonable at this time are 25 psi in the pre-stressed island area of the dam site (Area I on Plate 25), and 15 psi in the pre-stressed area south of the island (Area II of downstream dipping joints). These minimal design shear strengths are based generally on qualitative information obtained from bore hole photography and field examinations of the actual joint surfaces. Future detailed and more comprehensive studies may warrant favorable refinement of these shear values with resultant reductions in pre-stressing requirements and associated costs. Appropriate recommendations will be included in the Specific Design Memorandum for the dam. No tangible credit is taken in the current sliding stability analysis for shear through the pre-stressing cables or through any grout-filled seams along the low angled joints (See area grouting plan in paragraph 5.15) or for added resistance due to undulations on these joint surfaces. Intangible shear resistance from these and other sources add an important degree of conservatism to the overall stability analysis shown in this design memorandum and is considered sufficient to more than offset earthquake force considerations in arriving at a 4.0 safety factor.

5.14 Of the several types of intangible shear or frictional resistances studied, one of the most promising from the standpoint of future design usage is the added resistance to sliding due to undulations on the joint surfaces. An approximate quantitative determination of these undulations is definitely possible by use of bore hole photography and stereo net plots. For this and other reasons, a considerable number of bore hole photographs were taken at the dam site and the results thereof are presently under study. Preliminary finding from the bore-hole photo study indicate that undulations in hard rock do exist along the low angled joints, at least in the island area of the dam site (Area I). These undulations, which would ordinarily be expected to

occur in connection with the theorized tensional and "unloading" type origin of the joints, appear to average around 22 degrees in steepness of slope. A very preliminary theoretical analysis was performed to obtain a tentative quantitative determination of the additional sliding resistance due to any proven undulatory effect on an hypothetical low angled joint surface and is shown on Plate 14A. An approximate model study as demonstrated on Photos 21 through 23 has given a relatively close empirical confirmation to the sliding resistance formula derived on Plate 14A. Employing this formula and the above tentatively determined 22 degree average undulation value in the island area of the dam site results in almost doubling the natural sliding resistance. Should further comprehensive bore hole photography confirm the undulation angles on the major low angled joints throughout specific areas of the dam site, appreciable reduction in the proposed pre-stressing treatment will be possible in the final design (Figure 10A).

5.15 Depth of anchorage for the pre-stressing cables is based upon a depth requirement needed to engage a mass of quartz diorite bedrock below the lowest critical joint equal to the mass of rock and dam above this same joint or 25 feet penetration beyond the joint, whichever value is the lesser. In most cases this sets the depth of anchorage at approximately 60 feet depth below the natural rock surface. Because of the proclivity in this dam site for low angled planes of weakness, even the "plane" of anchor tips at their terminal depths will be stepped or otherwise caused to describe an irregular surface within the dam foundation (Figures 9 and 10). Prior to pre-stressing the dam foundation, an area-grouting program will be initiated to make sure all voids and open joints within the dam foundation are filled. This procedure will insure proper seating of the rock mass during the cable stressing operation and will thereby reduce associated cracking of the bedrock. Area grouting is also a prerequisite to insure that residual compression will exist across the joint planes after the pre-stressing is accomplished. The cost for this area grouting will be greatly reduced since the pre-stressed anchor holes, which would be already drilled on a grid pattern, could be utilized for the grouting. Each anchor hole used for grouting would then be redrilled after grouting, but before the grout had time to attain full set, thereby permitting the use of a drilled hole for two purposes. The combination of pre-stressing and area-grouting called for in Plan B will collectively yield an important additional benefit to enhance the overall dam site stability. This somewhat intangible benefit is the assurance that the dam site ridge will tend to act in a more monolithic manner and enable more efficient stress transfer from one point to another under future loading from the reservoir (and earthquake).

5.16 Because of the area grouting called for in Plan B, it is possible that the curtain grouting program for Plan B may be eliminated or sharply reduced in scope. Although this may result in significant saving in curtain grouting costs, it may also be necessary to increase the scope and corresponding cost of the foundation drainage program due

to the "tighter" bedrock conditions which are sure to result from the area grouting program (Plan B only).

5.17 Curtain Grouting.- Curtain grouting is planned for Plans A and C and, as mentioned above, to a much reduced extent for Plan B. The purpose of curtain grouting for all plans is to provide a definitive method of making sure that any undetected water passageways under or around the dam are tested and plugged by grout at pressures which can simulate reservoir head. Plates 15 and 16 show a section of the dam with the proposed grout hole depths and angles (Plan A and C). A grouting and drainage gallery is planned toward the heel of the dam as shown for both grouting and drainage purposes and for use as an inspection gallery during operation of the completed project. Preliminary curtain grouting estimates have been based on a line of grout holes five feet on center, angled upstream from the grouting gallery at a 3 vertical to 1 horizontal slope and penetrating a maximum of 75 feet of bedrock below the base of the dam. Grout take has been estimated at 0.15 sack of cement per linear foot of grout hole.

5.18 Foundation Drainage.- Because of the predicted low overall permeability of the bedrock in Plans A, C and particularly B, foundation drain holes will be spaced at least 5 feet on centers along the downstream edge of the grouting gallery and angled at a 30-degree angle off the vertical in a downstream direction. Average depth of each drain hole will be 50 feet penetration into bedrock. Since 33-1/3 percent reduction in uplift was assumed in the stability analysis for the concrete structures and 25 percent reduction was assumed for the stability planes in bedrock, it is important that a line of dependable drains be maintained during the life of the project (Plates 15 and 16). This provides further justification for a grouting-drainage gallery in the dam which will insure ready access for periodic drain hole reaming and inspection of the drainage system. In connection with this feature, uplift pressure build-ups under existing dams have been shown to increase over post-construction years due to accumulations of secondary mineralization within the water producing cracks in rock along the walls of the drain holes. Some consideration will also be given to the drilling of additional drains from the gallery at even flatter angles in order to provide drainage under some of the toe portions of the dam as well. This may become necessary to overcome an increase in uplift pressures under the downstream portions of the dam during winter months, when free discharge from seepage areas immediately downstream from the dam will be temporarily restricted or stopped because of freezing. Some consideration will also be given to providing a drainage relief gallery within the rock foundation at or near the location of the critical low angled joint on which uplift in stability analysis is based (Figure 11). This would insure a sufficiently low elevation for the datum relief plane (floor of drainage gallery) so that uplife reductions assumed in design would be assured. Such a gallery would further reduce the presently assumed 75 percent uplift,

and its inclusion will be thoroughly considered in preparing the specific design memorandum for the dam.

5.19 Future Design Memorandum for Dam.- A specific Design Memorandum on the dam will be submitted in December 1966. That memorandum will include more detailed and specific designs for both the foundation treatment and the dam structure. Further study as a result of 6-inch drilling, completed evaluations of bore hole photography, and more refined cost estimates, could possibly even show a combination of Plans B and C to be the most feasible development. The island area of the dam site, Area I for example, may be pre-stressed while the area of the dam site with downstream dipping joints (Area II) may be excavated and replaced with concrete. This finalized study will also have the benefit of an additional season of field work and will provide the technical basis for preparation of contract plans and specifications. Also included among next year's studies will be the economic and engineering feasibility of constructing a post tensioned dam at this dam site. This would be an added beneficial and supplemental feature to the basic Plan B pre-stressing scheme.

5.20 Reservoir Evaluation.- The basin in which Long Lake is situated is a completely rock-lined enclosure which has been deeply scoured by glacial erosion. There are thick accumulations of alluvial and glacial deposits at the upper end of the lake, and in all probability along portions of the bottom (See Photos 1 and 2). The proposed Long Lake reservoir, with a water surface some 80 feet higher than present Long Lake, will be contained in essentially the same rock-lined basin. Water in the proposed reservoir will be confined by relatively tight quartz dioritic bedrock through which only minor amounts of leakage are likely to occur. Equally important to this premise is the fact that the leakage cracks in the rock which do exist will never become appreciably enlarged by internal erosion or piping processes due to the durable nature of the rock. Low permeabilities of the confining basin rock will also reduce the volume of bank storage which will occur when the pool is raised, except in the upstream portions of the lake where extensive alluvial sands and gravel overlie the bedrock. No major slide problems are anticipated along the rock or overburden portion of Long Lake reservoir, although some minor adjustments in reservoir slopes will undoubtedly occur during and after pool raising.

SECTION 6 - LONG LAKE DAM

6.01 Selection of Type of Dam.- Design Memorandum No. 3, Selection of Plan of Development, described extensive studies undertaken to determine the recommended type of dam. Preliminary consideration was given to various types of dams including concrete gravity, multiple-arch, massive-head buttress, concrete-arch, rock-and earthfill structures. Of these the concrete-arch type was eliminated from consideration by the unfavorable configuration of the abutments, which would require massive thrust blocks and gravity wing walls on both abutments. The earthfill type was eliminated early in the studies of dam types because of the lack of suitable embankment material in any quantity in the vicinity. Because of the unlimited availability of excellent rock in close proximity to the damsite, greater consideration was given to a rockfill dam. However, due to the steep slope of Long River immediately below the lake outlet, and the even steeper slope of the lake bottom upstream, it became evident that this type of structure would be impracticable at this site. Thus, the three types of concrete structures remained for further consideration.

6.02 Studies of each type in sufficient detail to establish realistic cost estimates were carried out. For a height of dam greater than approximately 200 feet the massive, unreinforced concrete, multiple-arch dam appeared to have an advantage, mainly due to the lesser quantity of rock excavation required. However, for any lower height this advantage would not be realized because of the closer spacing of buttresses. For the massive-head buttress section, a maximum head width of 80 feet was indicated because of the possibility of cracking from shrinkage and from temperature stresses in a wider buttress head. Combined with the large base width required for stability in a buttress dam, this narrow buttress spacing resulted in considerably larger rock excavation quantities for this type of dam than for a concrete gravity section.

6.03 The concrete gravity dam was found to be lower in cost than other types considered. Because of inherent advantages in simplicity of construction and contractor familiarity, as well as minimum cost, the concrete gravity type was recommended for construction.

6.04 Selection of Height of Dam.- Design Memorandum No. 3 also describes the scoping analysis performed to accurately determine the optimum pool elevation of Long Lake. In this analysis the annual costs and benefits were determined for each additional 10-foot incremental increase in dam height. The scoping analysis resulted in the conclusion that development to a pool elevation of 895 provided for optimum development of the resource.

6.05 Selection of Axis.- The axis of the recommended dam has been located to utilize most advantageously the topography of the site, and is therefore angled in plan. This layout results in the maximum economy of construction, while assuring the stability of the structure.

6.06 Description of Recommended Dam.- The site plan, Plate 23, shows the dam and its relationship to other project features. The layout and details of the recommended dam, Plan B are shown on Plate 25. The dam will have a total crest length of approximately 800 feet, with a crest elevation of 905 and a height of 110 feet above the lowest point of the foundation. Crest width will be 12 feet. An ungated spillway, in two sections totaling 225 feet in length, will discharge into the existing bifurcated channel of Long River. The spillway crest elevation will be 895, corresponding with the maximum normal operating level of the reservoir. The 905 top elevation of the non-overflow sections will provide 1.5 feet of freeboard above the maximum flood pool elevation of 903.5. Information concerning the derivation of the spillway design flood is contained in Section 3, Paragraph 3.07. No vehicular traffic across the dam is contemplated, therefore spillway bridges are not provided. A grout and drainage gallery will be provided near the upstream face, closely following the base of the dam.

6.07 Alternative Plans Considered.- In addition to the recommended plan discussed above, two alternative plans have been studied in comparable detail. Plan A, shown on Plate 24, is identical to the recommended Plan B except for the method and degree of foundation treatment. Plan C as shown on Plate 26, differs from plans A and B in that the rock is excavated to the foundation joint planes, which results in a straight axis dam. The spillway is continuous rather than two separate sections as in Plans A and B.

6.08 Stability Analyses, Dam.- The stability studies of the dam above the foundation plane were of preliminary scope and were carried only to the point where structurally sound and reasonably economic sections were developed. In general, the procedures outlined in EM 1110-2-2200, 25 September 1958, were followed, except for changes in the shear friction factor equation and the elimination of the 0.65 sliding factor requirements. Design criteria and loading conditions are outlined in Figure 12. The results of the stability analyses are shown in Figures 13 through 18.

6.09 Stability Analyses, Foundation.- Because of the unusual foundation conditions at the damsite, stability studies of greater than normal scope have been required in order to determine the most economical method of foundation treatment. In these analyses the sliding shear friction equation as shown in Paragraph 3.04 of EM 1110-2-2200 has been used. These studies are summarized as follows for each of the alternatives considered:

a. Plan A: A minimum of rock excavation would be performed, with no special foundation treatment other than standard curtain grouting, drainage and surficial foundation preparation. Using the design shear values of 25 psi for the foundation joints in the island area and 15 psi for those on the right abutment (See Section 5), with a sliding friction factor of 0.65, a minimum shear friction factor of safety equal to 1.69 was determined (Figure 15A).

b. Plan B: Since the shear friction factor of safety for Plan A was less than the recommended value of 4.0, it was determined that an increase in the vertical force would be required to increase the shear friction factor under the given parameters. In order to furnish this large vertical force it was decided to use a method of foundation prestressing as shown in Plan B, Plate 25. The number of prestressing tendons was based on the following assumptions:

(1) Shear friction factor of safety of 3.0 for the earthquake loading condition, and 4.0 for all other loading conditions.

(2) Allowable shear stress of 25 psi in Area I, and 15 psi in Area II (See Plate 25).

(3) Sliding friction factor of 0.65 for all loading conditions.

More detailed discussion of the proposed prestressing system is contained in Section 5.

c. Plan C: The objective of increasing the shear factor of safety would be achieved by excavating the rock to the level of the lowest major low angled joint plane across the central two-thirds of the foundation.

6.10 Cost Comparison.- Cost estimates were prepared for each of the three alternative plans considered in sufficient detail to allow cost comparisons to be made. These estimates are shown in Table 6, and summarized as follows:

Item	Cost in Thousands of Dollars Plan A	Plan B	Plan C
Mob. & Demob.	275.0	275.0	275.0
Excavation and Foundation Prep.	56.1	56.1	291.2
Grouting & Drainage	144.5	181.7	152.0
Concrete	3,359.2	3,359.2	4,349.2
Steel, Reinf. & Misc.	39.0	39.0	40.2
Prestressing	None	772.2	None
Total Construction Cost	3,873.8	4,683.2	5,107.6
Order of Increasing Cost	1	2	3

6.11 It is apparent from the foregoing cost comparison that Plan A, which requires only minor amounts of foundation treatment, would have the lowest cost. However, it is not believed that sufficient shear strength would be developed in the low angled foundation joint planes to produce a satisfactory factor of safety. The increased cost of Plan B over Plan A is directly attributed to the addition of prestressing tendons and associated drilling and grouting. Plan C would avoid most problems connected with the foundation joint planes by removing all rock above such joints beneath the dam. The cost of this plan would be significantly higher than either Plan A or Plan B.

6.12 Recommendation.- It is believed that the inadequate safety of the minimum Plan A and the excessive cost of Plan C make the intermediate Plan B the logical choice for continued planning and design. It is therefore recommended that Plan B, substantially as shown on Plate 25 and described herein, be adopted. During preparation of the forthcoming specific design memorandum on the dam further studies will be made to more accurately determine the amount of prestressing required. Consideration will also be given to the economic feasibility of extending the post-tension tendons within the concrete dam.

6.13 Diversion Tunnel.- The diversion tunnel shown on Plate 27 is patterned after a method of lake-tapping much used in Norway. 1/ A tunnel will be driven toward the bottom of the lake from a point at a suitable elevation downstream from the lake outlet. A short distance from the piercing point a pit will be excavated in the tunnel bottom to accommodate the rock from the final plug. After careful investigation, both from the surface and from the tunnel, the final plug will be drilled and blasted. Either before or immediately after the final blast the control gate, subsequently discussed in paragraph 6.17, will be lowered to regulate the outflow from the lake.

6-14 The inlet of the 750-foot long diversion tunnel will be located at elevation 680, twenty feet below the power tunnel entrance and approximately 100 feet below the lowest foundation excavation for the dam. The diversion tunnel will have a horseshoe section 9.5 feet wide and 10 feet high, and a slope of 3 foot per thousand feet. Except for a short distance near the downstream portal the tunnel will be essentially unlined. A 6-inch minimum thickness of concrete paving will be placed on the invert, and the sides will receive a 2-inch nominal thickness of pneumatically applied mortar to improve hydraulic characteristics at low flows. The dimensions, slope and elevation of the diversion tunnel were determined by the requirement for passing the projected 10-year storm without raising the lake level enough to interfere with construction operations. The parameters so determined proved adequate to lower the lake surface initially in a period estimated at from two to three months.

6.15 Outlet Tunnel.- The Bureau of Reclamation, who will operate and maintain the project, has indicated a requirement for a permanent low-level lake outlet, for use in draining the lake for maintenance of the intake structure and trashrack. Studies have been made of the possibility of utilizing the diversion tunnel for this purpose. However, the requirement for an emergency gate and bulkhead as well as a regulating gate for any reservoir outlet other than a power conduit made this plan impracticable.

1/ Underwater Piercing of Lakes, by Chr. F. Groner, Water Power, 1 July 1960

6.16 As shown on Plates 27 and 28, and as described in paragraph 8.03, the outlet emergency gate, bulkhead and trashrack have been combined with the power intake structure. The concrete-lined outlet tunnel will be circular in section with a diameter of 9 feet throughout most of its 470 foot length, with a transition to a 9.5 by 9.5 foot horseshoe section where it joins the previously constructed diversion tunnel. The invert elevation at the inlet will be 690, ten feet below the power tunnel entrance.

6.17 Outlet Control Gate.- The gate for the regulation of discharge from the reservoir outlet tunnel will be placed at the downstream portal of the tunnel as shown on Plates 27 and 28. This gate and its associated structure and operating mechanism, together with the adjacent section of tunnel, will be constructed as part of the diversion works. The gate will be used initially to control the outflow during the draining of the lake.

6.18 The gate will be of the jet-flow type. It will consist of a leaf moved vertically on wheels, by means of a hydraulic motor, gear reduction unit, and a pair of threaded stems, with the leaf and surrounding housing shaped so that the water will issue from the orifice in a jet at all leaf positions. Power for operation of the gate will be provided by a portable diesel-driven hydraulic pump, which may also be used for operation of other gates on the project. Anchor bars, drilled and grouted into the rock surrounding the tunnel, will be used to resist downstream movement of the gate and control structure.

SECTION 7 - GEOLOGICAL EVALUATION OF POWER INTAKE WORKS

7.01 General Bedrock Character.- Explorations and studies conducted to date indicate the quartz diorite bedrock which will be encountered by the power tunnel, penstock tunnel, surge tank, and tailrace control structure, will be generally of good to excellent quality. As mentioned in Section 4, localized exceptions to this general premise will definitely occur at known major fault zones. These infrequent zones of highly broken and weathered rock have been identified with relative accuracy as to their precise position along the tunnel alignment as a result of strategic drilling in overlying fault zones (Photo 20) and detailed field mapping of the rock structures above tunnel grade. To the extent possible, all major structures have been located or aligned so as to minimize if not completely avoid known faults or closely spaced joint systems in the rock.

7.02 Power Tunnel Faults.- The 8,175-foot long power tunnel will cross three known major and several minor fault zones (Plate 20). A small shift in alignment of the tunnel from that shown in Design Memorandum 3 allows it to cross these fault zones more nearly at right angles, thereby minimizing required contact with fault zone materials. In the order of their anticipated occurrence along the power tunnel alignment and beginning from the upstream portal, the three fault zones are identified as "First Fault," "Second Fault" and "Glacier Creek Fault" (Plate 6A).

7.03 Nature of Fault Zones.- The central portions or "crush zones" in each of these faults are characterized by closely broken, mylonitized, and weathered rock, intermixed with definite lenses or stringers of plastic fault gouge. Adjacent to each central crush zone are border phases of less severely broken or weathered rock which, in turn, progressively grade back into the relatively fresh and unbroken main body of rock, completely untouched by faulting. Although all three fault zones possess generally similar types of lesser quality rock materials, the Galcier Creek Fault is by far the largest of the three. More extensive corrective measures are therefore anticipated in connection with tunneling operations through this particular fault zone.

7.04 Tunnel Water.- It is anticipated that the presence of water under relatively high pressures will be most likely but not limited to areas associated with faulting. Furthermore, it is expected that border phases of the major fault zones will be the largest producers of such high pressure seepage. This expectation is based on the premise that the fault-broken rock immediately adjacent to the highly weathered central crush zones will be more pervious because of lesser amounts of secondary weathering products filling the joints. Whenever water is encountered during tunneling operations, interceptor grout and/or drain holes will be drilled to either seal off or drain off this flow.

An exploratory "guide" hole will also be drilled in advance of the tunnel heading on approaching any of the major fault zones to signal the occurrence of high pressure flows or unfavorable ground conditions. Drilling of this guide hole will be phased with the blast hole drilling, shooting, and muck removal operations.

7.05 Other Remedial Measures.- Because generally good rock conditions are expected throughout a major part of the tunneling operation, it is not anticipated that an appreciable number of timber sets will be required. It is considered that requirements for timber sets will be limited principally to where rock conditions are closely associated with faulting. It is conservatively estimated that steel sets on a five-foot spacing will be required over approximately ten percent of the power tunnel length (Plate 20).

7.06 Rock Bolting.- A pattern of rock bolts will be placed from spring line to spring line across the arch of the power tunnel throughout a major portion of its length. For current estimating purposes, it is planned that the rock bolts will be approximately one inch in diameter, placed 4 feet on centers, and penetrate 6 feet into rock. Rock bolting will be performed immediately after tunnel excavation to minimize stress relief, overbreakage and rock falls.

7.07 Overbreakage.- In sound granitic rock, such as generally exists at Snettisham, proper use of conventional tunnel driving methods should control overbreakage to 20 percent or less. This percentage of overbreak is expected to be somewhat higher where the tunnel penetrates major fault zones. Utilization of pre-split blasting methods or special boring machines would minimize or possibly eliminate overbreakage. A relatively smooth circular bore would not only reduce rock bolting requirements by putting the tunnel perimeter in compression, it would also reduce the quantity and cost of concrete tunnel lining.

7.08 Design Rock Restraint.- Inasmuch as a greater part of the power tunnel length will be confined at a depth of several hundred feet in highly competent granitic bedrock, it is anticipated that this bedrock will ultimately provide seventy-five percent or more of the design lateral restraint required to resist internal hydrostatic tensile stresses in the tunnel lining. Full tensile reinforcement of the tunnel lining will be undertakin wherever poor ground conditions such as fault zones occur or where tunnel cover is inadequate (Plate 20). Adequate tunnel cover is presently based on the rule of thumb that whenever the cover in feet of good rock over the tunnel is less than three times the maximum internal tunnel head in feet of water, full tensile reinforcement will be provided in the tunnel lining. Where the rock is good quality quartz diorite as at the Snettisham project and the cover is adequate, this rule of thumb should provide a safety factor of approximately 4.4, based on consideration of the rock as a homogeneous and isotropic medium.

$$S.F. = \frac{250 \times 62.4 \times 3.0}{250 \times 2.75 \times 62.4 \times \frac{\mu}{1-\mu} \times 0.75} = 4.4$$

Where: 250 ft = Design Head
2.75 = Sp.Gr. Quartz Diorite
μ = Poisson's Ratio (Rock = 0.25)
3.0 = Cover-head Ratio
0.75 = Restraint by Rock

The Bureau of Reclamation considers adequate cover in highly competent rock to be 1.5 times the maximum internal tunnel head. If this criterion were used, only 14 percent of the power tunnel lining would need to be fully reinforced. This would provide a factor of safety of 2.2 based on the above theoretical approach. For the purpose of this design memorandum, the more conservative rule has been applied with the result that approximately 50 percent of the power tunnel lining has been designed as fully reinforced pressure conduit. The other 50 percent is designed with the lining absorbing 25 percent of the internal hydrostatic loading, and the competent granitic rock absorbing the other 75 percent. These internal stress design conditions adequately provide for reverse loading on the lining, which could occur from ground water pressures with the tunnel empty.

7.09 Geology of Other Waterways Features.- As stated in paragraph 7.01, current knowledge of the general project site geology has enabled many of the project features to be advantageously placed in specific areas of highely competent granitic bedrock. A few major structures such as the dam and power tunnel must accept limited zones of lesser quality rock because of their relatively fixed positions within the project scheme. Accordingly, the most detailed discussions on geological problems have been centered principally on these structures. No geologically related problems of a serious nature are anticipated with the intake structure, surge tank, penstock, or access adits. Except for the penstock, which will cross three minor fault zones at nearly right angles (Plate 21), generally high quality granitic rock is expected to be encountered. Information on recommended powder factors, shot hole patterns, firing delays, etc., will be submitted in future specific design memoranda for these structures. The close similarity in basic rock types between this project and Dworshak Dam in Idaho permits utilization of much useful analogous information on accomplished tunneling methods and costs.

7.10 Penstock and Access Adit.- For the purpose of this design memorandum, it is planned that rock bolting will be performed in the penstock tunnel and access adit in the same basic pattern and proportional quantities as outlined for the tunnel in paragraph 7.06. Overbreak conditions are generally similar to those discussed in paragraph 7.05. The steel lining of the penstock will be designed for 25 percent restraint by the surrounding rock except adjacent to surge tank and powerhouse or where rock cover is relatively light. In these

specific areas the steel lining will be designed for 100 percent of the internal pressure. As in the power tunnel, it can be generally stated that whenever the penstock lining is adequate for the internal pressure, it will also be adequate for any external pressures from ground water when the penstock may be empty.

7.11 State of Stress in Bedrock.- It is tentatively planned that the access adit to the power tunnel, just upstream from the surge tanks (Plate 30), will be driven prior to award of the major construction contract. This will provide the opportunity to observe an open bore and to perform at least a limited program of rock mechanics tests at or near the exact locations and depths of the power tunnel, penstock, and surge tank structures. There are several rock mechanics tests which, if performed in appropriate locations in the adit, can have an important bearing on the economics of tunnel lining and required penstock steel. In-place elastic constants of the rock are almost always less than those determined from rock cores and can be determined by standard jacking tests. These field values are the determinations which must be made in order to correctly ascertain the minimum amount of design restraint which can be expected of this rock, and to adjust steel and tunnel lining thicknesses accordingly. The principal field studies planned for the access adit work are: in-place jack tests to obtain typical field values for the modulus of elasticity (or compression), Poisson's ratio, and the 3-directional residual stress field existing in the rock body. In connection with this last feature, experience from other projects in granitic rocks and near major seismic zones (See Section 4.10) has sometimes shown the rocks at depth to be subjected to residual or "fossil" stresses up to several thousand psi. These unusually high stresses have in places been locked into the rock in past geologic times by glacial "unloading," mountain making processes or seismic load transfers through the earth's crust, and can be many times the stress magnitude that could be accounted for by the overlying rock load. Full advantage can sometimes be taken of this natureal pre-stressed condition within the rock if known to provide required lateral restraint.

7.12 The resultant strain from cutting a tunnel, penstock, or surge tank openings into highly stressed rock could cause the rock walls to move into the newly created opening. If a joint system were to exist at nearly right angles to one of the principal stresses, the rock walls could become unstable and slab off along such joints. Determination of joint systems and of principal stress directions could therefore, provide important information on preferred dirctions for making rock cuts and upon required positioning depth and spacing of rock bolts. The quasi-elastic flow of bedrock into all such openings has been shown on some other projects to decrease as a straight line when plotted along a logarithmic scale of time. It can be important to plot this rock strain against time in order to choose a time for concreting (tunnel linging, etc.) when the natural shrinkage rate of the emplaced concrete is equal to or greater than the inward rate of strain of the rock walls.

The original state of stress in the rock does not disappear, but is transferred back into the rock surrounding the opening which is better able to sustain the stress because of its unbroken and tri-axial state. If permanent support is put in too soon, this stress distribution will not be allowed to fully mobilize in the rock and the full stress will be transferred to the permanent supports and lining.

7.13 In addition to the standard methods of measuring these residual stresses in rock, such as strain rosettes, deformed bore holes etc., a relatively new method which has been proven successful in Australia and which employs the Freysinnet type jack is approximately as follows:

A slot is cut in the rock and allowed to partially close by the strain resulting from the locked-in stresses. A flat jack is then inserted into the slot and the rock is forced back into its original position as determined by 2 or more precisely measured points on either side of the slot. The unit stress required to jack the rock back into its original position is taken as the maximum residual unit stress acting in a direction perpendicular to the flat jack. Three mutually perpendicular slots or slots paralleling proposed excavation cuts are used to determine specific directions of the residual stresses.

7.14 Diversion and Intake Structure.- Plate 27 shows the respective locations for the upstream portals of the diversion outlet and power intake structures recommended in this design memorandum. These locations have been selected after extensive studies of lake bottom conditions by drilling barge, underwater TV camera, SCUBA diver-geologists, and leadline surveys (Photos 6, 10, 11 and 13). It will be noted that the upstream portal (underwater lake tap location) for the diversion tunnel is located on the right abutment rock slope approximately 300 feet upstream from the dam axis.

7.15 Studies have been made of an alternate diversion tunnel location adjacent to the power tunnel intake location at the dam site (Plate 22). The alternate location would be considerably shorter, but is not recommended because, (1) some 40 feet of overburden including large water-logged trees (Photos 12 and 14) would have to be excavated from present lake bottom, and (2) the dam foundation might be damaged from a relatively large uncontrolled blast in the last 20 feet of diversion tunnel required to effect the lake tap. It is considered advisable at avoid the possible propagation of shot shatter cracks up into the pre-stressed anchorage areas of the most critical portion of the dam site.

7.16 The presently selected location on the right abutment bare rock slope has an additional advantage in that it does not require mobilization of special barge-mounted excavating equipment to perform the deep and difficult underwater removal of debris from the present lake bottom.

7.17 Remedial rock treatment for the diversion and reservoir outlet tunnels will be similar to that discussed for the power tunnel in paragraphs 7.04 through 7.08. In addition to this treatment, an NX guide hole will be drilled in advance of the diversion tunnel heading as a constant precautionary measure. This will enable exact determination to be made of the last 20-foot critical section to be blasted in executing the actual lake bottom tap.

SECTION 8 - POWER INTAKE WORKS

8.01 General.- The power intake works include the intake structure, power tunnel, surge tank, penstock and appurtenant features. These structures constitute the waterways which convey the waters of Long Lake to the powerhouse. Their location and relationship to each other and to the remainder of the project is shown on Plate 3.

8.02 Intake Structure.- The intake structure, as described in the Project Document, would have consisted of a trashrack at the tunnel entrance and a vertical shaft at the shore of the lake to allow closing the tunnel by the use of stoplogs. During preparation of Design Memorandum No. 3, the Bureau of Reclamation indicated a requirement for placing the intake bulkhead as near to the tunnel entrance as feasible, to allow dewatering of the entire tunnel for inspection and repair. The revised intake structure shown in Design Memorandum No. 3, therefore, consisted of an inclined track on the slope of the reservoir, extending upward from the tunnel entrance to a bench at an elevation above the maximum flood pool. This plan was approved by the reviewing authorities. However, doubt was expressed concerning the location of the intake structure because of the difficulty of access, and the possible effect of snowslides.

8.03 During preparation of this General Design Memorandum, additional lake bottom soundings and investigations were made. These indicated the feasibility of an alternate site for the intake structure, immediately upstream from the dam, near the right abutment. As shown on Plates 27 and 28, the intake structure will include the bulkhead and emergency gate for the outlet tunnel, as well as the bulkhead for the power tunnel. Two trashrack panels, each 14.5 feet wide and 27 feet high, will be positioned in an extension at the front of the structure, covering the bell-mouth entrances of both the outlet and power tunnels. Each of these tunnels will be nine feet in diameter. Invert elevation of the outlet tunnel will be 690, that of the power tunnel 700. The reinforced concrete shaft which houses the gate and bulkhead slots will be 10 feet by 27 feet in plan, and approximately 215 feet in height. Also housed within the shaft will be the air vent pipes for each tunnel. The base of the structure will be founded on a bench cut into the rock, approximately 40 feet upstream from the face of the dam. Anchor bars will be drilled and grouted into the rock to anchor the base of the structure. Lake bottom sediment and debris will be excavated to elevation 675 in the area in front of the intake structure to allow free passage of the water into the trashracks.

8.04 The emergency gate and bulkheads will be operated by cable hoists, housed in an enclosure at the top of the shaft. The operating deck will be at elevation 905, level with the crest of the dam, and a 12-foot wide bridge will provide access. The gate hoists will be driven by hydraulic motors. Power for their operation will be provided by a portable diesel-driven hydraulic pump, which may also be used for operation of the outlet control gate. Provision will be made in the intake structure for a remote-reading reservoir water level indicator, connected by cable or radio to the powerhouse.

8.05 Power Tunnel.- Alignment of the power tunnel is shown on Plate 3, profile and details on Plate 29. The length will be approximately 8,175 feet, from the intake structure to the beginning of the steel penstock lining. Invert elevation will be 700 at the intake and 660 at the downstream end, with a slope of approximately 4.9 feet per thousand. The tunnel will be circular in section, with a finished diameter of 9 feet. This was determined to be the most economical diameter by studies similar to those described for the penstocks in Paragraph 9.07.

8.06 The lining will be reinforced concrete with a minimum thickness of 12 inches. Full tensile reinforcement will be provided wherever required by poor rock conditions or inadequate cover. Elsewhere, the tensile reinforcement will be designed to resist 25 percent of the hydrostatic head. Need for steel tunnel supports is expected to be limited to short sections in the vicinity of faults. However, rock bolts will be installed in the roof of the tunnel throughout most of its length to reduce rock falls and overbreakage.

8.07 Surge Tank.- The surge tank will be located at the downstream end of the power tunnel, as shown on Plate 30. Preliminary studies indicated that the restricted orifice type would be the most economical, principally because of the large reservoir drawdown which must be provided for. These studies determined the upper and lower limits of the maximum surge to be elevations 946 and 705 respectively.

8.08 As shown on Plate 29, the surge tank will consist of a 25-foot diameter concrete-lined shaft with a height of 250 feet. A 9-foot diameter steel-lined riser will extend from the penstock at elevation 660 to the bottom of the surge tank at elevation 700. At the top of the shaft will be an excavated bench at elevation 940. The top of the surge tank, with a reinforced concrete cover and air vents, will extend another ten feet to elevation 950. Provisions will be made for a remote-reading water level indicator in the surge tank, connected by cable to the powerhouse.

8.09 Butterfly Valve.- The butterfly valve, which will serve as the penstock emergency gate, will be located approximately 60 feet downstream from the surge tank riser. Its diameter will be selected to provide minimum hydraulic loses, while reducing the penstock diameter from 9 feet to 8 feet. The valve will be power-operated, and controlled from the powerhouse. An air inlet valve and a by-pass line for penstock filling will be included in the butterfly valve installation.

8.10 Valve and Access Vault.- The butterfly valve will be installed in a 26-foot by 30-foot concrete-lined vault. This vault will also inclose an 8-foot removable length of 9-foot diameter penstock, connected to the upstream end of the valve by an expansion joint. Removal of this section will allow full-section access to the tunnel and penstock for repairs and maintenance. A 24-inch manhole in this section will provide personnel access for inspection.

8.11 Access Adit.- Access to the valve and access vault will be provided by a 350-foot unlined adit. The adit will have a horse-shoe section 10 feet wide and 12 feet high, dimensions determined by the requirement for passing the 9-foot penstock sections and the butterfly valve. The adit floor will have a 6-inch concrete paving, and the roof will be covered with chain link fabric secured by rock bolts to provide protection from rock falls. The adit will be driven initially to provide access during construction and if possible to provide a site for a limited rock mechanics testing program. (See Paragraphs 7.11 thru 7.13)

8.12 Penstock.- Alignment and profile of the penstock are shown on Plate 30, and details on Plates 29 and 30. The optimum diameter of the penstock was determined to be 7 feet by the economic studies described in Paragraph 9.07. The profile was selected to provide a minimum of 200 feet of rock cover, except at the lower end where the penstock approached the powerhouse.

8.13 The penstock will have a total horizontal length of approximately 1,520 feet, beginning at a point 25 feet upstream from the surge tank riser and ending at the back wall of the powerhouse. Centerline length will be approximately 1,710 feet. Diameter will be 9 feet from the beginning of the penstock to the upstream end of the butterlfy valve. A 90^{o} tee section will connect the surge tank riser. Diameter will be reduced to 8 feet in the valve, with a further reduction to 7 feet in the compound bend at the upper end of the inclined section. Approximately 60 feet from the power-house a wye branch will be installed, with two 4-foot diameter penstocks continuing to the powerhouse.

8.14 The penstock lining at the upper end will be of type A-516 steel, with an allowable tensile stress of 17,500 psi. Minimum thickness of steel lining will be 3/8-inch. Type A-517 steel, with an allowable tensile stress of 28,700 psi, will be used where justified by increasing hydrostatic pressure. Maximum steel lining thickness at the powerhouse, using type A-517 steel, will be approximately 3/4 inch. Since pressure regulators will be used to limit pressure rise, design of the penstock lining will be based on a maximum of 20 percent overpressure from water hammer. In all cases where rock cover exceeds 200 feet the steel lining will be designed to resist 75 percent of the internal hydrostatic pressure. In areas of lesser cover near the surge tank and powerhouse, the lining will be designed for the full internal pressure.

8.15 Crater Lake Penstock, First Stage.- The Crater Lake surge tank and penstock will be located as shown on Plate 30. Approximately 200 feet of the Crater Lake penstock tunnel will be excavated simultaneously with that for Long Lake. At this point the two tunnels will be approximately 100 feet apart, a safe distance from which to commence excavation of the second stage portion of the Crater Lake penstock. Approximately 50 feet of the steel lining for this penstock, including the horizontal bend and the wye branch for the cross-connection to Long Lake turbine No. 2 will also be installed in the first stage, since construction of the powerhouse will preclude their later installation. It is anticipated that an adit will be constructed, beginning at a point near the switchyard, to intersect the first stage tunnel during construction of the remainder of the Crater Lake penstock.

8.16 For the purpose of this Design Memorandum, the previously selected diameter of six feet for the Crater Lake penstock has been used. However, during preparation of the specific design memorandum for the Long Lake power intake works additional studies will be made to determine the diameter and lining thickness required for the initial portion of the Crater Lake penstock.

SECTION 9 - COMPARISON OF ALTERNATIVE POWERPLANTS

9.01 General.- Previous planning has been based on construction of a conventional surface powerhouse, at the site proposed in the Project Document. However, the limited area available, the foundation conditions and the danger of snowslides at this site indicated the desirability of an underground powerhouse. Rock conditions appear favorable for such an installation. Studies have therefore been made of these two alternatives during preparation of this Design Memorandum. These studies were carried out in sufficient detail to provide realistic costs for each of several alternative layouts for both surface and underground powerplants, and to allow selection of the most favorable plan for further study.

9.02 Geology of Surface Powerhouse Sites.- Various sites for a surface powerhouse have been investigated with core drilling and seismic surveys (See Plates 17 and 18). Because of the difficulty in determining accurate bedrock contours in this area of fault valleys, steep natural drop-offs in rock surface, and large boulders overlying the true rock surface, much more exploratory effort than usual had to be expended on site investigations. Indications are that if known fault zones are avoided the occurrence of good competent granitic rock for powerhouse foundations near the immediate bedrock surface can be confidently predicted. In each case where true bedrock was penetrated during the powerhouse investigations, the rock was hard, fresh quartz diorite from the surface downward. Therefore the primary purpose of core drilling in siting the surface powerhouses has been to roughly identify the true bedrock surface.

9.03 Two surface powerhouse sites selected for more detailed study are designated as Sites A and B on Plate 17. Site A, almost the same site selected by the USBR in its preliminary studies, was investigated by five core drill holes. The bedrock at this site is covered to an average depth of 15 feet with unconsolidated material. Approximately 50 percent of the rock surface at Site B is exposed. As a result the preliminary investigations required at this site were reduced to two holes. Site B was found to be the better location because it is located closer to the surge tank, thereby reducing penstock length. It also requires less rock and overburden excavation and the necessity of concrete backfill is eliminated. Preliminary drilling has been completed for initial site selection, but additional exploratory work will be required during the 1966 field season.

9.04 Underground Powerhouse Investigations.- Investigations for underground powerhouse sites have been performed by the drilling of four core holes, approximately 1,000 feet in total length (Plate 19). From these and from rock studies performed for other features of the project, it can be predicted that the bedrock for any underground powerhouse site would be of good to excellent quality for this purpose. There are no anticipated adverse geological features that would require unusual contingencies in cost, or otherwise affect the decision as to the type of powerhouse.

9.05 For comparative studies, two underground locations as shown on Plate 38 were studied. Plan B would be located near the surface, with a minimum depth of rock cover and short tailrace tunnel. Plan C would be located immediately below the surge tank with the shortest possible penstock length.

9.06 Powerhouse Layouts.- Preliminary layout drawings for both conventional and underground powerhouses were prepared by the Hydro-Electric Design Branch of the North Pacific Division. These drawings formed the bases for preparation of comparative cost estimates. For the underground powerhouse plans, additional preliminary designs and cost estimates were prepared for access tunnels, for tailrace tunnels and for ventilation and cable tunnels which would be essential parts of such plans. Preliminary studies by H.E.D.B. indicate that pressure regulators would be required for the conventional powerhouse to provide acceptable speed regulation most economically. Similar requirements would exist for an underground powerhouse with penstocks of similar length. However, for an underground powerhouse located directly beneath the surge tank, the reduced length of penstocks would make pressure regulators unnecessary.

9.07 Penstocks.- It was apparent that the penstock costs would be an important point in the final location of the powerhouse. Studies were performed to determine the most economical penstock diameter for each plan. In these studies the total construction cost for penstocks of several different diameters were determined. These costs were then reduced to equivalent annual costs, using 3-1/8% interest and amortization for a 50-year period. Head losses for each size penstock were also determined and evaluated on an annual cost basis. The sum of these two annual costs for each diameter of penstock considered allowed selection of the most economical diameter, i.e., that which resulted in the lowest total annual cost. In all cases this economical diameter was closest to seven feet. Similar studies were performed independently by H.E.D.B., producing identical results. All further penstock studies were therefore based on a uniform diameter of seven feet. For the purpose of this Design Memorandum, the previously selected diameter of six feet for the Crater Lake penstock has been used.

9.08 For the conventional powerhouse two penstock layouts were considered: one with the penstocks located in tunnels, and the other with the penstocks on the surface. For the tunnel penstock layout a profile was selected which provided a minimum rock cover of 200-feet, except at the lower end where the penstocks approach the powerhouse. This layout is shown on Plate 30. For the surface penstock layout the profile would be horizontal from the base of the surge tank to the surface, then follow the slope of the mountainside to the powerhouse. Excavation of a trench would be required to provide as straight a grade as possible both to reduce the number of bends and anchor blocks required and to reduce head losses. The trench would accommodate both the Long Lake and Crater Lake penstocks. This layout is shown as Plan "A" on Plate 38.

9.09 As the penstocks for the underground powerhouse would also be installed in tunnels, they were sited so as to provide the shortest possible connection between surge tank and powerhouse. In each case the Crater Lake penstock was located in a separate tunnel, parallel to the Long Lake penstock. These layouts are shown as Plans "B" and "C" on Plate 38.

9.10 Tailrace.- For all plans considered an identical tailrace channel was assumed, providing a bottom elevation of (-) 1.0 at the upstream end. For the conventional powerhouse a control sill at this elevation determined the elevation of the powerhouse.

9.11 For the underground plans, a tailrace tunnel would be required. Considerable effort was devoted to determination of the most advantageous design for this tunnel, balancing excavation cost against head loss. A single horseshoe tunnel 17 feet in height and width and with a slope of 3 feet per thousand was selected, with separate smaller tunnels from each of the three unit draft tubes converging immediately downstream from the powerhouse. A sill at this point would control the minimum tailwater elevation. The length of the tailrace tunnel at the selected slope, plus the head loss at the control sill, determined the powerhouse elevation.

9.12 Cost Estimates.- Costs for powerhouse structure and equipment were developed from information furnished by H.E.D.B. These costs were adjusted to reflect transportation charges and the increased cost of on-site labor in Alaska. Costs for other items were developed from preliminary designs carried out in sufficient detail to provide comparable costs for each of the several alternative plans. Unit costs were developed by the same methods described in Section 20.

9.13 Economic Comparison.- Comparisons between the alternative powerhouse plans considered were based on the total economic cost of each, combining the construction cost with the present-worth value of total head losses. The alternatives considered are tabulated as follows:

Recommended Plan: Conventional Powerhouse, underground penstocks.

Plan A: Conventional Powerhouse, surface penstocks.

Plan B: Underground Powerhouse, minimum cover.

Plan C: Underground Powerhouse, maximum cover.

The Recommended Plan is illustrated on Plate 30, and Plans A, B and C on Plate 38.

9.14 The economic comparison of these four alternative plans is shown in the following tabulation:

	Cost in Thousands of Dollars			
Item	Rec. Plan	Plan A	Plan B	Plan C
Penstocks & Gate	3,221.0	3,418.3	2,480.0	1,817.3
Powerhouse & Equip	5,920.0	5,920.0	6,173.6	6,173.6
Pressure Regulators	274.9	274.9	274.9	None
Tailrace	31.4	31.4	496.0	1,282.1
Access & Cable Tunnels	None	None	555.2	1,485.1
Total Construction Cost	9,447.3	9,644.6	9,979.7	10,758.1
Order of Increasing Cost	1	2	3	4
Value of Penstock Head Losses	319.1	344.3	245.1	167.4
Value of Undeveloped Tailwater Head	135.2	135.2	321.0	430.9
Total Economic Cost	9,901.6	10,124.1	10,545.8	11,356.4
Order of Increasing Cost	1	2	3	4

9.15 The above tabulation shows that the order of increasing cost as determined by construction cost alone is not altered by the addition of head loss values. It also shows that while there would be a considerable saving in penstock costs by locating the powerhouse underground, this saving would not offset the additional cost of tunnel and powerhouse excavation. The cost of the surface penstock plan (Plan A) is shown to be less than that of the underground plans, though significantly greater than the cost of the recommended plan. The foregoing estimates, however, do not include maintenance costs. These would be higher for the surface penstock because of the periodic painting

which would be required. The additional problems resulting from ice formation in the penstocks and the danger of snowslides following the penstock trench and endangering the powerhouse would also make the use of surface penstocks undesirable. More detailed design studies and further refinement of cost estimates will very likely result in changes in the estimated cost of some of the individual features. However, since the studies for each of the alternative plans were carried out in equal detail it is not believed that the results of the economic comparison would be modified by further studies.

9.16 Recommendation.- It is therefore recommended that the conventional surface powerhouse with underground penstocks, essentially as contained in the Project Document and in Design Memorandum No. 3, Selection of Plan of Development, be retained for further planning and design.

SECTION 10 - POWER PLANT

10.01 General.- Powerhouse plans, basic data for cost estimates and text material concerning the power facilities in this General Design Memorandum were prepared by the Hydroelectric Design Branch of the North Pacific Division. Proposed plans contained herein cannot be considered final but do afford a sound basis for cost determinations. The number of units and kilowatt capacity of initial and ultimate installations are considered firm; however, further study will be required to select final type, size and setting of turbines and unit spacing. Results of detailed analyses of Snettisham power facilities and final recommendations for a firm basis of design will be contained in a future Preliminary Design Report on the powerhouse. Location of the powerplant with respect to other features of the project is shown on Plate 3. Powerhouse foundations are discussed in paragraph 9.03.

10.02 Basic Data.-

Plant capacity, initial, KW (nameplate rating)	40,600
Plant capacity, ultimate, KW (nameplate rating)	60,900
Type of turbine	Francis
Turbine rating, hp	32,000
Rating of generating unit, KW (nameplate)	20,300
Maximum pool elevation, Long Lake, feet	895
Maximum pool elevation, Crater Lake, feet	1,022
Minimum pool elevation, Long Lake, feet	720
Minimum pool elevation, Crater Lake, feet	828
Maximum tailwater elevation, feet	11.0
Minimum tailwater elevation, feet	(-)3.0
Elevation centerline turbine distributor, feet	4.5
Elevation bottom of draft tube, feet	(-)9.0
Diameter of penstock, Long Lake, feet	7.0
Diameter of penstock, Crater Lake, feet	6.0
Diameter of unit penstocks, feet	4.0
Spacing of main units, feet	34.0

10.03 Layout and Size.- The powerhouse will be a reinforced concrete indoor type structure equipped with a 75-ton bridge crane to serve for erection and maintenance of the units. Initially, the powerhouse will consist of two generator bays for the Long Lake units, a skeleton bay for the future Crater Lake unit, and an assembly bay, all complete with superstructure. The plans, sections and elevations of the powerhouse and space allocations and location of powerhouse equipment are shown on Plates 32 through 37.

10.04 The generator bays will be 48 feet 6 inches in width from outside to outside of the powerhouse walls, a width great enough to include the spherical valves so that the valves may be handled by the bridge crane. Generator bays one and two will have a total length of 68 feet 6 inches, the skeleton bay will be 44 feet in length and the assembly bay will be 45 feet 6 inches in length. The total length of the powerhouse will be 158 feet. A three-ton monorail hoist attached to the downstream exterior face of the powerhouse wall, complete with lifting beam and sling, will be used for handling the draft tube bulkhead. The auxiliary and control area will be located in the assembly bay.

10.05 <u>Turbines, Generators and Electrical Equipment</u>.- In accordance with the results of studies presented in Appendix A, each unit will be designed to provide a dependable capacity of 23,350 KW. The generators will be designed to produce this load at 115 percent generator nameplate rating at 0.9 power factor. The turbines will be of the vertical shaft, Francis-type with steel spiral cases and concrete elbow draft tubes. Each unit will be designed to produce a guaranteed output of 32,000 HP (23,350 KW) at minimum pool elevation. Best efficiency will be achieved at the average pool elevation. Spherical guard valves will be provided upstream from each unit for emergency shutdown and maintenance. In accordance with comments by the Bureau of Reclamation contained in their letter of 11 February 1965 (Exhibit 1), a valved connection will be provided between the Crater Lake penstock and the adjacent Long Lake unit to assure the operation of two units when the Long Lake waterway is shut down for inspection or maintenance. Preliminary studies indicate that pressure regulators will be required to provide acceptable speed regulation without the need for abnormal amounts of generator WR^2. The final selection of the type and size of guard valves and pressure regulators will be the subject of further study in the Powerhouse Preliminary Design Report.

10.06 The vertical generators will be 22,600 KVA, 0.9 PF, 13.8 KV at 60°C temperature rise with capability of operating continuously at 115 percent rated KVA. Individual three-phase, 13.8/138 kilovolt, 26,000 KVA power transformers for each unit will be located in a 138 KV switchyard near the powerhouse. Anticipated station service power scheme is a 13.8KV/480 volt, double-ended substation supplied from the generator terminals, with a diesel-driven emergency generator capable of supplying essential loads only.

10.07 Subject to completion of detailed studies, the anticipated layout of the electrical features of the powerhouse will conform to normal accepted practice for such plants. An auxiliary and control area is planned at the turbine floor elevation in the assembly bay where station switchgear, switchboards, control equipment and other electrical equipment will be located (Plate 34). Control features will be designed initially for centralized control from the powerhouse.

This system will accommodate modification to allow remote control from Juneau, should future developments make such an operation feasible.

10.08 Tailrace.- The minimum tailwater elevation will be maintained at minus 3.0 (approximate mean sea level datum) by an 80-foot control sill located 50 feet from the powerhouse as shown on Plate 31. The floor of the tailrace will slope upward from an elevation of minus 10 at the draft tube exits to the minus 3 elevation of the control sill. This area will be protected from erosion by reinforced concrete paving up to elevation 5, slightly above the average high tide level. The control sill will consist of a vertical concrete wall with a steel sheetpile cutoff to prevent underflow.

10.09 The tailrace channel, with a bottom width of 80 feet and a slope of one foot per thousand, will extend approximately 1,500 feet from the control sill into the tide flats. Bottom elevation will be minus 3.5 at the sill and approximately minus 5 at the seaward end. Side slopes will be 2 to 1 in the section between the powerhouse and the access road bridge, a distance of approximately 350 feet, and 3 to 1 in the remainder of the channel.

SECTION 11 - TRANSMISSION FACILITIES

11.01 General.- The major features for the transmission line and Juneau Substation are determined in this design memorandum and described in greater detail in Appendix B. The future specific Design Memorandum on the Transmission Facilities will include line profiles, structure locations, submarine cable location and further substation design.

11.02 Aerial photography and contour maps have been completed for the purpose of transmission route selection. The tentative route was located, marked in the field and clearing initiated. On Taku Inlet shallow sediment samples have been obtained, fathometer profiles run and a sparker survey is under way to obtain data for the submarine cable crossing.

11.03 Wind velocities were calculated for the transmission line from data recorded at Point Retreat on Lynn Canal. The maximum wind velocity from a 10,000-year return wind was 147 mph. Conductor icing was assumed not to be a problem on much of the line so the National Electrical Safety Code heavy loading of 1/2 inch of radial ice was specified from Juneau to Mallard Cove. From Mallard Cove to Snettisham 1 inch of radial ice will be used because of more severe climatic conditions. Maximum ice and wind loads are not expected to coincide. The relatively low-velocity-moisture-bearing winds come from the ocean where as the high-velocity winds come from the continent. Lightning levels are low along the transmission line. In Juneau the isokeraunic level is about 0.33.

11.04 Criteria for the substation and the transmission line were received from the USBR. Certain estimating data, specifications and drawings were also received and utilized.

11.05 Geology.- Most of the transmission structures will be founded on granitic type bedrock. There is only one type of foundation failure which could occur in these areas, the result of slab off on the steep slopes. Portions of the transmission line will cross muskeg areas. Where depths to firm material are excessive, special footings will be utilized. The submarine cable will require special investigations to determine a route which will avoid bridging the cable over rocks. The Sparker Survey being run by the United States Geological Survey should indicate these critical areas. The proposed location for the Juneau Substation is on the former Juneau garbage dump, two miles southeast of the heart of Juneau. Extensive investigation to determine the soil conditions in and under the dump are scheduled. Settlement of the fill and massive slide failure will be

investigated. There is also an active snow slide area just north of the selected site which may create high wind pressure on the substation structures. This slide potential will be investigated. Further geological investigations of the transmission line are scheduled for the summer of 1966.

11.06 Routes.- The route recommended in the Project Document was followed except for two sections. Plate 3 shows the proposed route with solid line and the Project Document route with a broken line. Studies indicated that the cost of the additional length of the proposed route was more than offset by the added construction, maintenance and outage costs associated with the higher elevations of the more direct route.

11.07 The submarine cable route tentatively crosses Taku Inlet at the point where the transmission line will be the shortest and the cable length will also be a minimum. Further submarine exploration will determine whether minor adjustments of the presently planned alignment will be required.

11.08 Voltage.- The voltage recommended for the transmission line is 138 KV. After extensive investigation of voltages from 69 KV through 230 KV, 138 KV was calculated to have the lowest overall cost including debt service, value of losses, interim replacements and operation and maintenance expenses. Early in the study, the 69 KV line was eliminated because of excessive losses. The 230 KV line was found to require too high charging current and much more expensive cable. The change in voltage from that specified in the project document results from an increase in energy production, the longer transmission distance and the increased value of losses.

11.09 Conductor.- The 795 MCM ASCR conductor proposed in USBR criteria was found to be the most economical choice for the line. An analysis was made of different conductor sizes, their cost, their effect on structure cost and the value of the losses. These were all present worthed to obtain the most economical conductor. The 795 MCM conductor has considerably more capacity than is required but the high value of losses dictated its choice.

11.10 Structures.- Wood H-frame structures will be used over 39.3 miles of the line. The remaining 5.3 miles will be steel or aluminum towers. Wood poles are used wherever feasible because of their lower cost. In the 5.3 mile area near the powerplant, metal towers will be used because of space limitations, difficulty of installing guys, strength, adaptability to steep slopes and the ability to maintain them by use of small helicopters. This 5.3 mile area is very steep, has long spans, and is at a higher altitude than the rest of the line as well as being inaccessible from the beach.

11.11 Taku Inlet Crossing.- Taku Inlet will be crossed by four, single-conductor, 138,000-volt submarine cables. One cable would serve as a spare and will be paralleled with one of the other cables under normal operating conditions. Taku Inlet reaches a depth of about 680 feet at the cable crossing. Each cable will rest on the bottom and will be about 14,250 feet in length.

11.12 An economic study of various cable types and sizes showed that a 250 MCM oil-filled, paper-insulated, lead-sheathed, steel-armored cable was the best alternative. This cable would have the capability of carrying 113 MVA, however, it is the smallest size recommended for this voltage by standards for manufacture. The excess capacity will permit the addition of other power plants to the system if required at a later date. An overhead crossing was also investigated, but higher costs, less reliability, hazard to aircraft and other factors led to early elimination.

11.13 Juneau Substation.- The recommended substation plan, essentially the same as that outlined by the USBR, is shown on Figure 2, Appendix B. Triple-rated, auto-transformers with a maximum capacity of 40 MVA are recommended. When the second transformer is added, it will be paralleled with the first with isolating disconnect switches provided to facilitate maintenance.

11.14 The substation will also be provided with a 3,096 square-foot building which will house offices, control room, garage and warehouse. Space will also be provided for outdoor storage (Plate 39).

11.15 Snettisham Switchyard.- Initially, the switchyard at Snettisham will consist of two transformers, two transformer bays and one line bay and will generally conform to USBR construction standards (Plate 31).

11.16 Communications.- The communications planned for the construction period will consist of four voice channels and one teletype channel on a VHF, UHF or microwave radio link. This system will be adequate for the operation of the project. Communications will be covered in greater detail in a Communications Supplement to Design Memorandum No. 5, "Access and Construction Facilities."

11.17 Maintenance.- Inaccessibility of large portions of the line and ruggedness of terrain will cause operation and maintenance costs to be much higher than the average costs for lines of similar voltage located in other states. Much of the equipment required for line maintenance will be used only for emergency repairs and will be used rarely during normal operations. The line can be patrolled

most economically by helicopter. Roads are only practical for about 26 miles of the line. The other 20 miles, would have to be maintained by other means, probably by helicopter. The design of the structures will be such that they can be constructed and maintained with a 4,000 pound capacity helicopter. Storage space will be provided at Juneau Substation and at Snettisham for various items of line maintenance equipment, poles and other spare parts.

SECTION 12 - ACCESS AND CONSTRUCTION FACILITIES

12.01 General.- This section describes proposed development of facilities for access to and within the project area. These include the dock and floats and permanent and temporary access roads. Discussions of residency and camp facilities and of fire control requirements are also included. Diversion works are described in conjunction with the dam in Section 6. Locations of the features discussed herein are shown on Plate 3. Wherever possible, facilities required during the construction period will be so designed and located that they can be utilized to the maximum degree in development of permanent facilities required for the operation and maintenance of the project. The design and location of many of the facilities associated with construction activities will, within broad limits, be left to the discretion of the contractor.

12.02 Docking Facilities.- Transportation of materials, supplies and personnel to the project will be provided solely by sea and air. Facilities for the unloading of boats, barges and seaplanes will therefore be essential to operation of the project. These facilities will be located in a protected cove on the southerly side of Star Point approximately two miles south of the powerplant. Explorations in the dock area showed foundation materials to be organic silt and fine rock flour having low strength and buoyed up with high artesian pressures. Because of the low strength and the susceptibility of the area to underwater sliding, all dock structures will be located near shore where piling can rest on bedrock.

12.03 A barge dock suitable for the unloading of medium-craft barges will be constructed adjacent to the shore with a 40-foot by 350-foot timber deck at elevation 15. The maximum tidal range in Speel Arm is about 28 feet. Therefore, dredging of an area in front of the dock to an elevation of approximately minus 23 will be required to allow entrance of barges at minimum tides.

12.04 Adjacent to the barge dock, floats will be provided for the loading and unloading of seaplanes and of boats up to approximately 60 feet in length. The boat float, anchored by timber guide piles, will be 60 feet long and 16 feet wide and will be supported by foamed-plastic logs. The seaplane float, secured to the end of the boat float, will be 30 feet wide and 40 feet long.

12.05 A landing craft will be utilized for transportation of equipment and materials for transmission line maintenance and also for moving vehicles and heavy supplies between Juneau and Snettisham. A timber ramp 50 feet wide, with a slope of 20 percent, will be provided for loading and unloading this craft. The upper end of the ramp will adjoin the deck of the barge dock and will undoubtedly be

utilized extensively during project construction. In addition, the ramp will be suitable for beaching of amphibian aircraft for loading and unloading during periods of unfavorable weather.

12.06 Additional facilities in the dock area will include a small, electrically-operated crane with a capacity of approximately 4,000 pounds; storage facilities for gasoline and oil in 55-gallon drums; a small shelter building with a telephone connected to the project system and area lighting to provide for night operations.

12.07 Permanent Access Roads.- An access road having a 24-foot roadway and a gravel surface will be constructed to connect the dock area with the powerplant and camp area. It will be about two miles in length and will extend across the upper end of the tideflat on a gravel fill. In this area the roadway surface will be at its minimum elevation of 15. In the vicinity of Crater Creek the road will rise to a higher elevation in a rock cut section, then descend to a fill section at elevation 15 across the tideflats in the powerplant area. Three bridges of steel and concrete construction will be required for this road. Single-span bridges of 88 and 45 feet will be used at Crater Creek and Glacier Creek respectively, and a four-span bridge totaling 155 feet will be used to cross the tailrace channel. Corrugated metal culverts will be provided for cross-drainage where required.

12.08 A similar road will be constructed from the camp area to a staging area near Long River at approximately elevation 200. This road will generally follow the easterly edge of the braided stream channel of Glacier Creek on gravel fill, then rise to the saddle above First Lake in a rock cut section. From this point to the staging area the road will follow the westerly side of the valley of First and Second Lakes on a gentle grade with a combination of cut and fill sections. Throughout its length the permanent access roads will have a maximum grade of ten percent and a maximum curvature of 57 degrees.

12.09 Foundation conditions are not expected to be critical except in the tideflat area between Crater Creek and the powerplant. In this section the foundation is composed of organic silts and fibrous peaty soils having variable and relatively low strengths. Based upon vane shear tests and observations, the maximum fill height has been set at 14 feet which provides a factor of safety against sliding of 1.2. Other sections will be founded either on bedrock or granular materials and should present no stability problems. In the Second Lake area, the road alignment crosses an area of varved silt. No stability problems are expected but excavated slopes will require stabilization to prevent erosion from heavy rains. Granular material for embankments is available at convenient locations along the alignment.

12.10 Temporary Access Roads.- Access from the Long River staging area to the Long Lake damsite, as well as access to tunnel portals and

surge tank, will be the responsibility of the contractor and will not be provided by the Government. Provisions will be made in the contract documents to control the construction of all temporary roads in order to satisfy the requirements of the U. S. Forest Service. Prior to termination of the final construction contract, a review of all the then-existing roads on the project will be made in conjunction with representatives of the Bureau of Reclamation and the Forest Service. All roads which are judged suitable for project operation will be retained. All other roads and trails will be restored as nearly as possible to their original condition and treated to prevent erosion, as specified by the Forest Service.

12.11 Residency Facilities.- Temporary facilities for the operations of the Resident Engineer's staff are proposed for early construction and will be included in the first construction contract. A firm location for these facilities has not been selected, but it is tentatively planned that they will be in the area which will be graded for the contractor's camp, adjacent to the existing Government camp. The Resident Engineer's office building will contain 3,200 square feet. An 800 square foot field laboratory and 800 square feet of storage space will be provided, either in the same building or in a separate building connected by a covered walkway. In addition, warm storage space for six Government vehicles will be provided.

12.12 Government Camp.- The existing Government camp, constructed for use during the pre-construction planning phase of the project, will be retained for use by Government personnel during project construction. It may be made available for temporary use by the first construction contractor during preparation of the contractor's camp. The existing camp is a self-sufficient facility providing accommodations for 36 men together with a warehouse and a small office. Modification of the existing water supply system will be required with a connection to be provided from the contractor's camp. Electric power and operational services for the Government camp will be provided by the contractor.

12.13 Contractor's Camp.- An area suitable for the contractor's camp is available on the easterly side of Glacier Creek adjacent to the permanent access road and the existing Government Camp. The area is a gravel hill partially covered with timber and overlain with from three to five feet of peat. Suitable material excavated during construction of the camp will be used as fill for the access road. Clearing and grading of the camp area, together with construction of a water supply system and a sewage disposal facility, will be included in the first project construction contract. Installation of structures; water distribution and sewage collection systems; power supply and distribution system for his own use will be the responsibility of the contractor. After completion of the project, the camp area will be available for release to the Forest Service for possible recreational development.

12.14 Fire Control.- Because the area encompassed by the Snettisham project lies within the boundaries of the Tongass National Forest, special fire control precautions during construction will be necessary. An annual fire control plan will be prepared by the Forest Supervisor of the Forest Service in agreement with the Corps of Engineers, and this plan will be binding on the contractor and all sub-contractors.

SECTION 13 - BUILDINGS, GROUNDS AND UTILITIES

13.01 General.- Buildings and grounds development in the project area will be limited to those facilities required for operation and maintenance. Because of the isolated location of the site, more extensive provisions for maintenance and repair will be necessary than would be required for comparable projects in other areas. Reinforced concrete fire resistant construction will be used for all project buildings. While all buildings will be of utilitarian design, the principles of good architectural design will not be neglected. Heating will be all electric and, during the course of further studies, the use of a combined heat-pump system for the powerhouse, dormitory and office-shop building will be investigated.

13.02 Housing and Service Area.- The dormitory and the powerhouse office and shop building will be constructed in proximity to the powerhouse as shown on Plate 31. The area upon which these buildings will be located, together with the switchyard area on the opposite side of the tailrace channel, will be used initially as a disposal area for materials excavated from the tunnels and powerhouse. Additional fill material from borrow will be used to complete the grading of the area.

13.03 Housing.- On the basis of criteria outlined in ER 415-2-301, it is believed that the provision of permanent housing for project employees is justified. The nearest community to the site is Juneau, 28 miles distant by air and 45 miles by water. No roads exist or are proposed to connect the site with Juneau. In the Project Document it was proposed that fifteen dwellings be constructed for the use of permanent project employees. Current planning by the Bureau of Reclamation for operation of the project, however, indicates that personnel will be based in Juneau. Transportation from Juneau to the site will be provided by the Government on a regular weekly shift basis. Therefore, no family housing will be provided and a dormitory will furnish accommodations during the regular tour of duty.

13.04 As illustrated on Plate 39, the 8,500 square foot two-story dormitory will have a 20-man capacity with individual bedrooms and semi-private baths. It will include kitchen, dining, lounge and recreation facilities together with adequate food storage and refrigeration. An underground passageway will connect with the office and shop building and with the control room level of the powerhouse. By this means the three buildings will be formed into an integral complex, facilitating traffic between them and assuring all-weather availability of off-duty personnel in emergencies.

13.05 Powerhouse Office and Shop Building.- Office space for the project superintendent, together with shop and storage facilities,

will be provided in a 3,500 square foot building immediately adjacent to the powerhouse. The machine shop in this building will be adequate in size and equipment to allow for all except major powerplant repairs. Storage space will be included for a stock of spare powerplant equipment. A vehicle storage and service area will allow for the storage of three vehicles. It will include a vented grease pit, vehicle maintenance tools and equipment and space for spare parts. In this area will be located a 125-KW diesel-driven generator for emergency power supply.

13.06 Transmission Maintenance Building.- A 2,250-square foot building, located as near to the dock area as possible, will provide storage for transmission line maintenance equipment, materials and supplies. It will be capable of storing a line truck, D-6 tractor, tracked snow vehicle and a hot stick trailer. Warehouse space will be adequate for all materials and supplies requiring covered storage, and a gravel surface storage yard will be included.

13.07 Grading, Seeding and Planting.- Because of the heavy rain and snow-fall in the project area, care will be taken in design of all graded areas to preclude maintenance problems resulting from flooding or erosion. Gravel surfaced drives and parking areas will be provided for all areas requiring vehicular access. A limited amount of seeding and planting will be done in the housing and service area to enhance the architectural appearance of the buildings. Consideration will also be given to the possibility of seeding of excavated slopes in areas of easily erodible materials.

13.08 Utilities.- Domestic water may be supplied either from surface or underground sources. Several small streams exist in the vicinity of the powerhouse and groundwater is plentiful. A reliable source will be developed during subsequent studies, and treatment and storage facilities will be provided as required. Sewage disposal facilities will also be provided in appropriate areas as required.

13.09 Electric power will be supplied by the station-service system in the powerhouse, with a diesel-driven standby generator for emergencies. A three-phase, 440-volt power line will provide power for the transmission maintenance building and the dock area. An interconnecting project telephone system will provide communications between all the buildings as well as with the dock area.

SECTION 14 - CONCRETE AGGREGATES

14.01 Availability of Construction Materials.- The Snettisham project has both advantages and disadvantages with respect to availability of suitable construction materials. Relatively high quality natural deposits of sands and gravels have been laid down by past stream deposition and glacial outwash in many areas within short distances of the project (see Section 4). The ruggedness and unusual steepness of the terrain in this area, however, has tended to enter very strongly in determining the economic suitability of specific concrete aggregate sources. Further complicating this study are the long distances between various major concrete structures of the project as well as the required sequence of construction (Plate 6A).

14.02 Sources Studied.- Although many initial sources of concrete aggregate have been studied to some degree, three basic deposits have entered into definite final consideration for proposed use at the project (Plate 6 and Photos 15 though 19). These areas of study are Source A, a natural deposit of glacial outwash sand and gravels at the head of Long Lake; Source B, a natural deposit of sandy oversized gravels in the alluvial fan of Glacier Creek; and Source C, a ledge rock source of manufactured aggregate just off the left abutment of the dam site.

14.03 Sources Compared.- Cost estimates and overall economic comparisons of the three concrete aggregate sources are shown in Appendix C. Inasmuch as unit costs per yard are relatively close, especially for Sources A and C, there could well be other factors which may ultimately determine final usage of these construction materials. A partial list of these factors follows:

a. Results of trial mix design studies which will be performed by NPD testing laboratory on samples from Sources A and B. A trial mix study has been performed for Source C with generally very satisfactory results.

b. It is relatively much easier to obtain 6-minus material for the mass concrete in the dam Sources B or C as compared with Source A. The cement factor in the mix could be reducted with coarser aggregate sizes in the mass concrete, but workability would be correspondingly decreased with the angular 6-inch material from Source C.

c. It is highly desirable to have rounded-particle, natural aggregates for the extensive pumpcrete operations for lining over a mile and a half of tunnel. By far the best material for this work would be from Source A at the head of Long Lake. This same factor would also apply, but to a lesser degree, to the placing of other

structural concrete where smaller natural aggregate sizes are preferred in areas where forms are tight and reinforcement steel is closely spaced.

d. Results of preliminary concrete aggregate tests demonstrate that for physical quality, the aggregates are to be ranked in the approximate order, Source A, C and B. Beyond a certain point these differences may be only academic as long as all sources are capable of producing materials which attain minimum prescribed test values.

e. Source C is most conveniently located for the dam, intake structure, diversion structure and upper part of the power tunnel. Source B is most conveniently located for the powerhouse, penstocks, surge tanks and lower portion of power tunnel.

14.04 <u>Testing Programs</u>.- Appendix C shows the results of some of the aggregate tests performed to date on materials from Sources A, B and C. Also shown are the results of the trial mix study on manufactured aggregates from Source C. The only test complete result not yet available from the trial mix study of Source C is the mortar bar expansion test, but from preliminary indications the results of this test should be favorable. It is not anticipated that alkali reactivity will be a problem with any of the aggregate materials at Snettisham. Therefore, it does not appear that it will be necessary to specify the use of low alkali cements. Of interest and of possible future usefulness is the close similarity in test results from Snettisham Source C and test results from Dworshak Dam in Idaho where manufactured granitic ledge rock is currently being utilized.

14.05 Some of the sand fractions from Source B have fallen below standard strength in the "mortar making properties" test. This deposit, which is near tidal level shows evidence of higher degree of weathering, probably resulting from humic acids in the nearby peaty topsoil as indicated by bron limonitic coatings on many aggregate samples. As a result, only the gravels will be obtained from this source and the sand fraction will be scalped off. A deposit of apparently suitable sand does exist within a few hundred yards of Source B proper (Plate 6).

14.06 Full-scale trial mix studies will be underway this winter at the NPD testing laboratory on materials from Sources A and B. These studies will include 7, 28 and 90 day breaking tests on concrete test cylinders, flexure tests on test beams, mortar bar expansion test, and other standard tests conducted on the aggregates themselves. The testing program will also include thermal studies on the coarse aggregates for future mass concrete temperature studies. These latter tests will include conductivity, diffusivity, and thermal expansion tests on the coarse aggregate from Sources A and B. It is anticipated that useful related information of this type for Source C can be obtained by close examination of the test results and theoretical thermal studies already performed for Dworshak Dam.

14.07 <u>Sources Adopted for this Report</u>.- For the purpose of preparing cost estimates for this design memorandum, Source C, the manufactured aggregate, is utilized for the dam, intake structure and reservoir outlet tunnel. Source B, the Glacier Creek deposit, is utilized for powerhouse, surge tank and power tunnel. Other economic and physical factors relating to site logistics and proven aggregate qualities may change this presently proposed usage to some degree. These important considerations will be thoroughly covered along with the final results of all testing programs in later specific design memoranda. Firm recommendations on individual or combined usage of the various available concrete aggregate sources will then be made.

SECTION 15 - RESERVOIR MANAGEMENT AND PUBLIC USE

15.01 General.- This section of the General Design Memorandum presents information on the resources existing in the project area and those that may be created by the formation of the pool and related works, along with the responsibility of management of any natural and recreational resources, existing or formed. The data contained in the following paragraphs are summarized from the Preliminary Master Plan (Design Memorandum No. 4 dated April 1965).

15.02 Scenic, Historic and Archeologic Resources.- The project areas lie in a very rugged and almost completely unpopulated region with no existing roads and is accessible only by boat or float plane. The climate, typical of Southeastern Alaska, is characterized by moderate temperatures at sea level, mild winters, cool summers and heavy precipitation mostly in the form of rain. Dense forest cover with accompanying thick undergrowth extends from sea level to a maximum altitude of approximately 2500 feet, but these timbered areas are often broken by large muskegs which are not conducive to tree growth.

15.03 The entire southeastern area is a mountainous mass intricately cut by waterways which separate the islands from each other and from the mainland, affording intermittent passage far into the Coastal Range. These seaways are deep, many over 400 feet, and have rocky bottoms. Configuration of the land mass provides one of the most scenic areas of the continent. Many peaks on the mainland rise to between 6,000 and 8,000 feet or higher while many of the islands have peaks 4,000 feet or higher. On the mainland, glaciers fill many of the narrow canyons and attain great size in the northern part of the area. Most of the higher peaks, both on the mainland and on the islands, also contain glaciers.

15.04 A colorful history has been forged in Alaska by the white man's effort to expand the European influence to the North American Continent. Russia claimed what is now the State of Alaska and much of the west coast by reason of early exploration and later attempts at colonization. During the middle eighteenth century, Russian ships pursued fur seal and sea otter herds in coastal waters and engaged in extensive fur trade with the Indians. Efforts at colonization were made to exploit more effectively the fur trade of the region. In 1799 the Russian American Company was organized to administer Alaska and to promote discovery, commerce and agriculture and to propagate the Russian Orthodox faith. Count Baranof, the company's first executive, extended the sphere of Russian influence from Bristol Bay to northern California. Sitka, then the capital, became a colorful and cosmopolitan port where ships from many nations came to trade. In 1818, however, the Russian Navy assumed control and reversed the policy of free trade. Russia then attempted to monopolize all trade within the area by

closing the coast north of 51° latitude to all but Rissian ships. Shortly thereafter Russia was weakened by wars and internal trouble and permitted her influence in Alaska to decline during the following half century. Then, rather than lose the Territory to another power, Alaska was sold to the United States in 1867.

15.05 Prior to purchase of Alaska, Americans had entered the region to trade, prospect for gold or to fish and by the late 1860's were fishing Alaskan coastal waters for cod and salmon. After the first cannery was built at Klawak, Prince of Wales Island, in 1879, the industry grew steadily and by 1899 the annual salmon pack reached a million cases. In the meantime prospectors were coming into the Southeast to search for minerals--principally gold. Their success is evidenced by the large lode gold mines which were operating on Gastineau Channel before 1900. Thus the two basic industries, fishing and mining, were well established by the turn of the century. Many communities came into existence to serve as supply and processing centers for fishermen. Other communities, notably Juneau and Douglas, developed as mining camps.

15.06 Southeastern Alaska not only possesses an abundance of natural resources upon which to base a tourist industry, but is also rich in historical associations of a colorful past. Early tribal life of the Indians is manifested in totem poles, grave houses, kitchen middens, community houses and native arts such as basketry, wood carving and blankets. Some of the best examples of these may be found in the vicinity of Sitka, Wrangell and Ketchikan. In Sitka, former seat of Russia's colonial government, evidences of the occupation are widespread. Many of the structures built by the Russians are still in use, among which is the Cathedral of St. Michael, a Russian Orthodox Church, completed in 1848. The Indians, as well as the Russian traders, American and Canadian trappers, did very little penetrating of the small rivers along the mainland such as Speel River. For the most part, their activities were located on main navigation channels and islands. There was little activity in the Gastineau Channel area until the discovery of gold-bearing quartz and rich gravels in the Gold Creek Basin in the summer of 1880. Development of the large, low grade ore bodies was rapid and the Juneau area soon became the most important producer in Alaska. In 1900 the seat of government was officially transferred to Juneau from Sitka, although the actual move was made gradually over several years.

15.07 Fisheries.- The present economy of Southeast Alaska is largely based on its extensive fisheries industry which furnishes the livelihood for the large majority of the population either directly or indirectly. The waters of Southeast Alaska are prolific producers of fish, especially the protected waters which lie between the hundreds of islands which make up most of the coast line. The offshore waters are also important producers and have in recent years shown a stable and steady growth in their yield. The principal types of fish caught in the region are salmon, halibut, herring, sablefish and shellfish.

15.08 The salmon found in Southeast Alaska are of five species which in their order of importance are the pink, chum, coho, red and king. Pink and chum salmon usually spawn in lower tributary or coastal streams often within a mile or so of the ocean, whereas kings and cohos prefer the larger streams and travel much farther. The reds use only streams with lakes in their watersheds and spawn along the shores of the lakes or in the tributary streams above. The canning of salmon in Alaska was inaugurated in 1878, and by 1889 there were a dozen salmon canneries in Southeastern Alaska. Their 1950 canned salmon pack amounted to a little over 1,187,000 cases and had a value of about $26,000,000 at the packing plants. The 1963 pack amounted to 1,216,145 cases at a value of $28,123,360.

15.09 Wildlife.- In Alaska wildlife is classified in three main categories--big game, fur animals and birds. The most abundant big game in Southeast Alaska is the Sitkan black-tailed deer followed by bear, mountain goat and moose. The deer, which are estimated to number about 40,000 are found in some 12,000 square miles of range extending from Dixon Entrance to Cross Sound. The brown and grizzly bears, highly prized by big game hunters, are most numerous on Baranof and Chichagof Islands and the black bear is found on Kuiu, Kupreanof, and Prince of Wales Islands and along the mainland coast. The offshore islands were devoid of these animals until some were successfully transplanted to Baranof Island by the U. S. Fish and Wildlife Service. The mountain goat, by its way of living in remote and inaccessible areas, is assured protection from all but the hardiest of human hunters. The few moose that inhabit this region are confined to the mainland coastal areas.

15.10 Fur animals have played an important part in the development of Southeast Alaska since its early settlement by the Russians. Many hundreds of the native Indians and numerous professional white trappers and old time residents depend on hunting and trapping and the selling of furs as a mainstay for their existence. The principal fur bearing animals are mink, marten, beaver, land otter, muskrat and weasel. Wolves, though not strictly classed as fur animals, have pelts of sufficient value to warrant their mention. The average annual fur take for three years, 1948 to 1950 inclusive, was valued at more than $450,000, 80 percent of which was mink.

15.11 Large numbers of migratory water fowl and upland game birds propagate in the area. Many kinds of ducks breed here, but the most common are the pintail, mallard and American wigeon. An isolated colony of the beautiful large Pacific eider duck nests in the Glacier Bay National Monument area. The whitecheeked goose is predominant on the many islands of this region.

15.12 The most abundant upland game birds are grouse and ptarmigan. Ptarmigan, the Alaska State bird, is a form of grouse which nature has

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endowed with the ability to change its coloration between summer and winter periods. Alaska grouse, including ptarmigan, are subject to epizootic disease and periodically die off in large numbers. These birds increase to great abundance every 8 to 10 years and then almost vanish, after which the flocks slowly build up again from scattered survivors.

15.13 Economy.- The five most important sources of income in Southeastern Alaska are government, fishing, forestry, tourism and mining. The fishing and forestry industries have formed the major part of the economy of the area. However, the Juneau area's economy is based on Federal, State and local governments. Various agencies of the governments provide jobs for approximately 1,500 people and represents a payroll in excess of $8,300,000.

15.14 Fishing and Fish Processing.- In 1950 the fishing industry employed almost half of the workers within the area at the peak of the season. The main activity is concentrated within a few months period, and off-season employment falls considerably below that during the peak. Salmon and halibut fisheries now catch the maximum numbers permissible in the interest of conservation, but the industry has some opportunity to expand. Many streams and lakes now barren of salmon runs could be opened up to spawning and could produce large new runs if channel obstructions were removed and, in some cases, ladders were constructed to aid fish past waterfalls. Some further expansion is possible through greater utilization of bottom fish, shellfish and material now wasted.

15.15 Timber Resources.- Nearly all of southeastern Alaska, including the Juneau area, is within the boundaries of the Tongass National Forest. Total land area of this forest is approximately 16 million acres. The estimated saw-timber volume in this area exceeds 143 billion board feet, surpassed by only three other states: Oregon, Washington and California. The stand consists principally of western hemlock, Sitka spruce and lesser amounts of cedar. Due to steep slopes, commercial timber grows in relatively narrow bands along shorelines of the mainland and island, rarely extending inland more than five miles. The vast network of navigable waterways make much of the timber readily accessible. The Forest Service has estimated that 90 percent of this timber lies within three miles of tidewater. Approximately 3 million acres of the Tongass National Forest is presently estimated to contain timber of merchantable quality. Average volume per acre of these stands is between 25,000 and 35,000 board feet, but volumes exceeding these are common over wide areas. The majority of merchantable trees are from 20 to 40 inches in diameter and from 85 to 150 feet in total height. Additional development of timber resources promises to give a tremendous boost to the Juneau area.

15.16 Recreational Resources.- From a recreation standpoint, scenery is the foremost tourist attraction. The coastal area of Southeastern Alaska lies within a portion of an immense semicircle of mountains that extend from the southern extremity of the State to the Aleutian Chain. Hundreds of square miles of towering snow-capped peaks have created a rugged terrain broken further by long arms of the sea extending inland. The resulting fjords provide views as spectacular as any in the world. Adding both interest and magnificence to the scenery in this area are the tremendous glaciers, several of which extend to tidewater and are among the most spectacular on the North American continent. A few, such as the glacier of Le Conte Bay, break off in enormous blocks to become icebergs. Alaskan glaciers are particularly interesting in that they are remnants of the extensive ice field which once covered much of the North American continent.

15.17 Another important attraction for the recreationist is the wildlife resource of the region which affords good hunting and sport fishing. The area contains a variety of game animals including big game, wild fowl and fresh and saltwater fish.

15.18 Because of its remoteness, the Snettisham project will initially offer very little recreational value to the poeple of the region. Crater and Long Lakes are such that no substantial recreation use would be attracted to them. However, interest in the power plant area at tidewater may attract some visitors via water transportation. At the present there are an estimated 400 people a year that visit the Speel River area in pleasure craft for the purpose of sport fishing.

15.19 In view of the quality of construction and depth of dredging proposed for the project docking facilities, it is possible that the Snettisham project be made an occasional stopover point on Alaska's Marine Highway System. Since the State's ferries draw only about 18 feet of water, they will easily be able to enter the project harbor at any half-tide or better; and they could layover for as much as twelve hours permitting tourists to use available roads, trails and camp facilities to obtain views of the magnificent scenery and to visit project features. Land areas along the immediate shoreline will be available for recreational use, such as camping and picknicking, with access principally by boat and amphibious aircraft.

15.20 Views and Interests of Other Agencies.- Representatives of the U. S. Forest Service and Bureau of Reclamation have visited the Snettisham site, both powerhouse and reservoir areas, to consider the recreational values and development possibilities. It was determined that although the area has only limited national or regional recreation significance and boat landing facilities should be maintained and a suitable campsite provided for recreational use.

15.21 U. S. Forest Service.- In accordance with the provisions of the Memorandum of Agreement by the Secretaries of the Army and Agriculture relative to management of land and water resources at water development projects of the Corps of Engineers located within or partly within the National Forest, Appendix B of EM 1130-2-302, the U. S. Forest Service and the Corps of Engineers will cooperatively plan the development of facilities needed for recreation, wildlife and other similar purposes and which would be compatible with the primary project functions. Management of the land and the use and development of resources, including water oriented recreation, will become the responsibility of the U. S. Forest Service. It is mutually agreed that since the entire project lies within the U. S. Forest Service boundaries, that since the U. S. Forest Service has many existing recreation sites in operation in Southeastern Alaska, that additional sites are under construction and improvement and that the U. S. Forest Service has the qualified personnel and organization readily available to assume this responsibility, it is to the best interest of the U. S. Government, State and regional area to allow the U. S. Forest Service primary jurisdiction over any proposed recreational development in this area.

15.22 Bureau of Reclamation.- Funds were appropriated in fiscal year 1964 for initial preconstruction planning of the Snettisham project. In accordance with the March 1962 agreement between the Department of Interior and the Department of Army, The Corps of Engineers will design and construct the project which, upon completion, will be operated and maintained by the Bureau of Reclamation. The Bureau of Reclamation has agreed that the U. S. Forest Service be authorized the responsibility of operating and maintaining all project features pertaining to public needs for recreation, wildlife and other uses compatible with the primary purpose of the water storage facility. The Bureau's prime responsibility will be to operate and maintain the reservoirs, penstocks, powerplant and transmission lines including the sale of power produced.

15.23 Development Considered Necessary.- Development by the Corps of Engineers will be limited to these facilities that are to be constructed as part of the powerhouse and those needed for construction purposes. Facilities for visitor viewing inside the powerhouse will be part of the original design and construction of the structure. A small boat dock, to be used for construction purposes, will be left intact for future use of recreational boats with an adjoining stagging area to be cleared and leveled for camping purposes. Any other development will be by the Forest Service on a programmed basis based on the overall needs of the entire Juneau area and the many recreational sites operated by the Forest Service.

SECTION 16 - REAL ESTATE

16.01 General.- The main project area is all Government owned land and consists of approximately 14,022 acres which includes the environs of two alpine lakes, Crater and Long Lake, together with an area of the Speel River flats suitable for a power generating station and housing facilities. An additional area will be required for the location of a high-voltage single-circuit transmission line which will carry project power to the load center at Juneau. Two general routes have been considered both extending from the project area south to Mallard Cove after which one route would go overland to Taku Inlet which it would cross by submarine cable to Bishop Point thence on into Juneau. The alternate route would be the same except that at Mallard Cove it would follow the coastline, instead of crossing overland, around to Slocum Inlet. Due to the construction and maintenance problems associated with the higher elevations reached by the more direct overland route, the alternate route along the coastline is being recommended. This right-of-way will be approximately 44.6 miles long and 200 feet wide containing some 1,083 acres. A general description of the recommended transmission facility is contained in Section 11. From its point of origin at the project site to Bishop Point the right-of-way is located entirely upon Government owned land with a possible exception at Taku Harbor where it may cross a small portion of private land. From Bishop Point to the Juneau Townsite Boundary, a distance of approximately 4½ miles, the right-of-way is located sufficiently inland to avoid all but some 180 feet of private land. The distance from the Townsite Boundary to the termination of the transmission line near Juneau is approximately 5 miles, the ownership of which is held by private interests and the State of Alaska. The right-of-way extends approximately 900 feet across privately owned small tracts, 18,600 feet across numerous mineral surveys and mining claims most of which are patented and all owned by Alaska-Juneau Industries, Inc., and 14,900 feet across State land. This State land has been selected and granted tentative approval which is tantamount to ownership. The mineral surveys and claims are lying idle and have not been in production for several years. As near as can be determined there are no other such interests in either the main project area or the remainder of the transmission line right-of-way. The Juneau substation will be situated on a 400' x 400' site approximately one mile southeast of the City. Two tentative locations were investigated, however, one of them, the A-J Industries' rock dump, will not be considered further due to the high cost of acquisition. The other location is the abandoned City garbage dump which the City has indicated it would consider making available for use as a substation site. The estimated total acreage for the entire project is approximately 15,109 acres and is broken down as follows:

ACRES

	FEDERAL	STATE	PRIVATE
Main Project Area	14,022.00		
Transmission Line R/W	923.00	69.00	91.00
Juneau Substation			4.00
Totals	14,945.00	69.00	95.00

Grand Total 15,109.00

16.02 Type of Acquisition.- The Corps of Engineers is to acquire all the land and rights necessary for the Snettisham Project and is to construct said project after which it will be turned over to the Department of Interior, Bureau of Reclamation, for operation and maintenance. Land in the main project area, totaling approximately 14,022 acres, together with 923 acres in the transmission line right-of-way is under jurisdiction of the Forest Service, U. S. Department of Agriculture. An application for the withdrawal of the 14,022 acres has been made with the concurrence of the Forest Service. The transmission line right-of-way on Federal land will be acquired by use of Forest Service Construction and Use Permits. A Memorandum of Understanding has been drawn up by the Forest Service wherein they grant permission to the Corps of Engineers to occupy all the National Forest land necessary for planning and construction of the project while they, the Forest Service, retain jurisdiction over as much of the described lands as are not to be occupied for project purposes. Acquisition of the transmission line right-of-way across State and private lands will be accomplished by acquiring permanent easements. Since structures of a permanent nature are to be constructed on the substation site this land will be acquired in fee.

16.03 Relocations.- There will be no relocations of roads, railroads, pipelines, utilities or cemeteries, etc. The principal roads to be constructed will be in the main project area and will connect the dock area with the powerplant and camp areas, a distance of around two miles. There will also be some access roads to the transmission line. The camp will be connected by road to a staging area near Long River.

16.04 Resettlement.- None.

16.05 Real Estate Costs

a. Fee.- No land is to be acquired in fee other than the Juneau substation site. Original interest had centered on a portion of the old Alaska-Juneau Industries' rock dump which is well located and would probably meet foundational requirements. This rock dump comprises

around 80 acres, is located a short distance from downtown Juneau and has water frontage. The owners of this land intend to develop it into an industrial park, therefore, they are quite reluctant to sell any part of it and would strongly contest any condemnation action. This land is undoubtedly of considerable value as it is about the only remaining land available for such use in the Juneau area. No comparable sales were found, however, it is assessed at full value by the Borough Assessor at $.60 per square foot which would give a value indication for the 160,000 square feet of substation land of around $96,000. For these reasons further consideration of this property has been discontinued. Present consideration is being given the abandoned City garbage dump, an area of around 7 acres located approximately 3,000 feet southeast of the rock dump. The City has tentatively indicated it would make the land available at no cost, however, this has not been formalized. The market value of this land is considered to be substantially less than the rock dump site due to location and possible development restrictions caused by unstable ground materials. Its value is estimated at around $25,000 as compared with the rock dump site which, once it begins to become industrially developed will no doubt increase values in this area. A more complete study will be made for Design Memorandum No. 11, Real Estate, to be issued at a later date.

b. Easement.- Of the total 1,083 acres in the transmission line right-of-way, it will be necessary to acquire permanent easements covering approximately 160 acres. This includes 69 acres of State land and 91 acres of private land. All but 5 acres of the private land consists of mineral surveys and claims owned by A-J Industries, Inc. The 5 acres involves the crossing of around four small tract ownerships of 4 to 5 acres each and a privately owned U. S. Survey tract. The actual right-of-way location has not been finalized. It is the policy of local power companies in both Anchorage and Juneau to acquire this type of easement for no consideration unless damages are incurred by the property owner. In this case approximately 100 feet of the right-of-way will be kept clear of timber and brush for purposes of maintenance and non-interference. Even though the transmission line should be sufficiently above the road to be out of sight its presence and the clearing of timber may still result in a loss of value to the private land amounting to as much as full value for the land involved. No sales of this land were found, however, they are valued at around $500 per acre by the Borough Assessor which appears reasonable. The mining claims have been the property of A-J Industries for many years and have not been offered for sale even though they are not being operated. The Assessor values them at $400 for a standard claim of around 20 acres or $20 per acre. The State land has not been valued by the Assessor and in all probability easements can

be acquired across it for little or no consideration. For purposes of this report, however, it will be valued at $25 per acre taking into account the small amount of merchantable timber involved.

c. Administrative Costs.- These costs are based upon an estimate of the actual work to be performed in acquiring the necessary real estate rights. Included are mapping and surveying, appraising, title evidence, negotiating and closing. They are listed as follows:

MAPPING AND SURVEYING		$4,000
APPRAISING		500
TITLE EVIDENCE		350
NEGOTIATING AND CLOSING		650
	TOTAL	$5,500

d. Summary.- The following is a summary of the estimated real estate costs as of this time. Since the availability of the sub-station site at no cost to the Government is not presently known its value will be included. A contingency factor of 25% is also included to cover the possibility of having to acquire easement rights over additonal private land and to consider a possible increase in value from the date of this report to final project acquisition.

LAND RIGHTS

GOVERNMENT OWNED LAND - 14,945 acres (INCLUDES 923 ACRES TRANSMISSION LINE R/W)	NO COST
STATE LAND - EASEMENT RIGHTS ONLY 69.00 ACRES @ $25/ACRE	$1,725.00
PRIVATE LAND - EASEMENT RIGHTS ONLY 86 ACRES MINERAL SURVEYS @ $20/ACRE	1,720.00
5 ACRES SMALL TRACT OWNERSHIPS @ $500/ACRE	2,500.00
JUNEAU SUBSTATION SITE - FEE ACQUISITION	25,000.00
TOTAL COST OF LAND RIGHTS	$30,945.00
CONTINGENCIES - (25% of $30,945)	7,736.00
ADMINISTRATIVE COSTS -	5,500.00
ESTIMATED TOTAL OF REAL ESTATE COSTS	$44,181.00
CALLED	$44,200.00

16.06 Acquisition Schedule.- Application has been made for the withdrawal of Federal land in the main project area and approval is now pending. A Memorandum of Understanding between the Alaska District Engineer and the Regional Forester involving all the Federal land needed for planning and construction of the project is currently under study. It is presently the plan to have the transmission line right-of-way and substation site acquired by December 1966 and no later than March 1967.

SECTION 17 - DESIGN AND CONSTRUCTION SCHEDULE

17.01 General.- A condensed schedule for design and construction of the first stage of the Snettisham project is shown in Figure 20, based on the appropriation of $2,500,000 in construction funds for fiscal year 1967. Previous project planning has been based on appropriation of initial construction funds in fiscal year 1966, with first power to be available in December 1969. The demand for power in the Juneau area is increasing steadily; additional diesel generation must be added to the present system within the next year, and further additional generation will be required in 1970; and the recent announcement by the U. S. Forest Service of an extensive forthcoming timber sale in Southeastern Alaska indicates the probability of construction of a large wood and pulp processing plant in the Juneau area by 1971. These are all factors which emphasize the importance of maintaining the previously scheduled date of December 1969 for initial power generation at Snettisham. The present schedule, therefore, has been developed with that objective. however, it is recognized that the time available is limited and that this will be a difficult schedule to maintain. Critical path scheduling studies are currently underway, with preliminary results to be available shortly. It is believed that with the aid of this modern tool of management and with the full cooperation of reviewing authorities, the proposed schedule can be acheived.

17.02 Design Schedules.- The scheduled submission dates for specific design memorandums and for contract plans and specifications have been revised in accordance with the revised construction schedules described below. The Preliminary Design Report on the powerhouse, together with the plans and specifications for the powerhouse and power equipment, will be prepared by the Hydro-Electric Design Branch of the North Pacific Division. Design Memorandum No. 10, Power Intake Works, and the plans and specifications subsequent thereto, will be prepared jointly by an Architect-Engineer and the Alaska District. All other design work will be performed in-house by the Alaska District, with review by a Board of Consultants.

17.03 Total engineering and design costs (exclusive of preauthorization funds) are estimated at $3,200,000 for the first stage and $4,400,000 for the completed project. Of this amount $2,005,000 has been made available through fiscal year 1966 to date. The remaining requirement will therefore be $1,195,000 for the first stage, and $2,395,000 for the completed project.

17.04 Construction Contracts.- Present planning is based on construction of the major portions of the project in two contracts, Contract A9 will be primarily an excavation contract, including excavation of all tunnels and the surge tank, and the rough excavation for the powerhouse. It will also include the diversion-outlet control structure and gate, docking facilities, access roads, site preparation for the contractor's camp, tailrace channel and powerhouse area grading. A final item of particular importance to orderly completion of the project, will be the foundation grouting and installation of prestressing tendons for the dam. Contract A9 is scheduled for award at the end of April 1967, and for completion in September 1968.

17.05 Contract B9, the second major construction contract, will consist primarily of concrete work. It will include the dam, lining of all tunnels and the surge tank. Construction of the powerhouse, installation of powerhouse equipment, the switchyard, and construction of all buildings at the site will also be in this contract. Award of contract B9 is scheduled for November 1967 and completion for February 1970. It will thus overlap contract A9 by about 10 months, and both contractors will be at work on the site during the construction season of 1968. It is recognized that this situation is likely to cause problems, particularly if the two contracts are awarded to different contractors. Consideration will be given in the CPM studies currently underway to the possibility of combining both into a single contract.

17.06 The transmission facilities will be constructed in three separate contracts. A contract for clearing of the right-of-way will be awarded in April 1967, and completed in June 1968. A second contract for the submarine cable section of the transmission line will be awarded in December 1967, and completed in October 1969. Because of the unusual nature of this work, and the long lead-time involved in the manufacture of the cables, it is considered advisable to place this item in a separate contract. The contract for the overhead portion of the line, including the Juneau substation, will be awarded in April 1968, and completed in February 1970.

17.07 Several procurement contracts will be required for the powerplant and miscellaneous equipment, with schedules depending upon manufacturing lead-time and on installation schedules. The earliest award dates, for turbines and bridge crane, will be contingent upon appropriation of construction funds and upon approval for advertisement prior to allocation of such funds. These contracts are tentatively scheduled for award in October 1966.

17.08 Construction Schedules.- Construction schedules are discussed in the above paragraphs concerning construction contracts. More detailed schedules of the individual components of these contracts are shown in Figure 20. Upon completion of the previously discussed CPM studies, a revised and more detailed construction schedule will be submitted by separate action.

17.09 Funding Requirements.- In accordance with the foregoing discussions, fund requirements by fiscal years for the first stage of the Snettisham Project are as shown in the following tabulations:

Funds available prior to FY 1966	$ 1,205,000
Appropriation, Fiscal Year 1966	800,000
Requirement, Additional, FY 1966	400,000
Requirement, Fiscal Year 1967	2,500,000
Requirement, Fiscal Year 1968	13,000,000
Requirement, Fiscal Year 1969	14,200,000
Requirement, Fiscal Year 1970	7,700,000
Requirement, Fiscal Year 1971	495,000
TOTAL	$40,300,000

SECTION 18 - OPERATION AND MAINTENANCE

18.01 General.- The project will be operated and maintained by the Bureau of Reclamation. Preliminary studies by that agency indicate that all employees will work a regular tour of duty from Monday through Friday, with the exception of the operators whose tours will vary in such a manner as to provide 24-hour attendance. All maintenance men will normally be stationed at the powerplant from Monday morning through Friday afternoon. Commercial amphibious air transportation will be provided by the government each Monday morning and Friday evening on a scheduled basis between Juneau and Snettisham.

18.02 The tentative organization of the staff, as presently proposed by the Bureau of Reclamation, is as follows:

Supervisory

1 Project Superintendent
1 Administrative Assistant

Powerplant	Transmission
6 Powerplant Operators	1 Lineman
1 Powerplant Mechanic	1 Groundman
1 Powerplant Electrician/Lineman	1 Heavy Equip Operator/Bargeman
1 Meter and Relay Mechanic	1 Boat Captain

Camp

1 Cook

As shown above, the staff would consist of 16 men. In the event of an emergency, either the powerplant or transmission maintenance crews would be qualified and prepared to assist the others in restoring service.

18.03 Operation, Maintenance and Replacement Costs.- The annual operation and maintenance costs are estimated by the Bureau of Reclamation to be $390,000 for the first stage and would not increase with the addition of the second stage. The costs of interim replacements are estimated to be $86,000 for the first stage and $94,000 for the completed project, based on information furnished by the Bureau of Reclamation. Annual OM&R costs would therefore be $467,000 for the first stage and $484,000 for the completed project.

SECTION 19 - COORDINATION WITH OTHER AGENCIES

19.01 Bureau of Reclamation.- Under terms of the authorizing act, the Department of Interior will be responsible for operation and maintenance of the Snettisham project upon its completion. The Bureau of Reclamation of that Department has been designated as the responsible agency, therefore, close coordination has been required and maintained between the Alaska District offices of the Bureau of Reclamation and the Corps of Engineers. The Bureau of Reclamation has furnished functional design criteria for the transmission facilities, as well as all available background material relating to the pre-authorization studies. Numerous conferences and exchanges of data and information have taken place during the course of studies leading to preparation of this design memorandum. All previously submitted design memorandums have been reviewed by the Bureau of Reclamation, and their comments thereon have been incorporated where applicable.

19.02 Forest Service.- The project will be located almost entirely within the boundaries of the Tongass National Forest. Management of the project lands and the use and development of resources, including water oriented recreation, will become the responsibility of the Forest Service upon completion. Criteria for land clearing, spoil disposal, and road and trail construction has been or will be determined by the Forest Service. Coordination has been maintained through individual contact and correspondence concerning these matters, as well as project land withdrawals and transmission line location. Previously submitted design memorandums have been reviewed by the Forest Service, and their comments thereon have been incorporated where applicable.

19.03 Fish and Wildlife Interests.- During preparation of the Project Document, the U. S. Fish and Wildlife Service reported that it was believed that the project would not materially affect the fish or wildlife resources of the area. Alaska Department of Fish and Game concurred in these findings. Following initiation of preconstruction planning, both agencies were again contacted. This General Design Memorandum will be transmitted to both Federal and State agencies for comment.

19.04 Federal Power Commission.- The San Francisco office of the Federal Power Commission has been kept informed of the general status of the Snettisham project through correspondence and telephone contacts. In August 1965 the Chairman and Chief Engineer of FPC, together with the Regional Engineer visited the project site and were

briefed on the planning studies underway. The FPC furnished power values for use in scoping and economic analyses. This General Design Memorandum will be transmitted to the Federal Power Commission for review and comment.

19.05 Other.- Various other Federal, State and Local agencies have been contacted informally during the course of planning studies. Bonneville Power Administration shared results of their extensive experience in construction and operation of transmission lines including salt water exposure and submarine cable installations. The U. S. Coast and Geodetic Survey furnished information on survey monuments and bench marks. The U. S. Geological Survey cooperated in the establishment of a tide gauge, and in the gathering of hydrologic data. The Soil Conservation Service assisted in the establishment of snow courses and the U. S. Weather Bureau installed a climatological station on the site. The U. S. Bureau of Land Management, the State Division of Lands and the City of Juneau cooperated in the real estate studies described herein.

SECTION 20 - PROJECT COSTS

20.01 Scope of Estimates.- The estimates in this memorandum includes both initial and completion phases of project construction. The former includes the Long Lake dam, power intake works, the powerplant with the two Long Lake power units, the transmission facilities, and all general features of the project. The latter includes the Crater Lake intake works, the third power units and appurtenant equipment.

20.02 Estimates.- Project costs are summarized by features in Table 3, wherein costs of temporary construction items are distributed to project features and proportionate basis as shown in Table 4. The detailed estimate shown in Table 5 includes costs for both stages and lists construction quantities and unit costs applied to the various features of the project.

20.03 Bases for Estimates.- The majority of the unit costs shown were developed from detailed studies of the labor, materials and equipment required for the various items of work. For certain items for which full design details were not available, unit costs were developed from bid prices for other projects in the Pacific Northwest. All unit costs based on projects in other areas were converted to estimated Snettisham Project costs by applying varying factors, averaging approximately 1.7, which includes the effects of increased transportation cost, higher wages and cold weather construction requirements. Costs for turbines, generators and other powerplant equipment were furnished by the Hydro-Electric Design Branch of the North Pacific Division. These costs were adjusted to include transportation costs and the increased cost of on-site labor.

20.04 All costs in this design memorandum are based on estimated price levels of October 1965. No escalation of prices during the construction period has been provided for in the estimates. The contingency allowance added to the cost of the various features varies from 15 to 25 percent, and averages about 20 percent for the total project.

20.05 Lands and Damages.- The costs of lands and damages were divided between the features 01. "Lands and Damages," and 07.80 "Transmission Plant-Lands and Land Rights." Since all of the project site area and the major portion of the transmission line route lie within the boundaries of the Tongass National Forest, land costs will be nominal. The Juneau substation and a short portion of the transmission line are within the Juneau townsite, and acquisition from private interests will be required when final locations of these features have been determined. Real estate costs are explained in detail in Section 16, and summarized in Tables 3 and 5.

20.06 Comparison of Present Estimate and Latest Approved Estimate.- The estimates are compared in the following tabulation and differences are discussed in the following paragraphs:

First Stage Development
(cost in $1,000)

Cost Acct No.	Feature	Present Estimate Oct 1965 Base	Latest Approved Estimate July 1965 Base
01.	Lands and Damages	3	20
04.-	Dams	16,698	19,435
.1	Main Dam & Spillway	(6,463)	(6,061)
.3	Outlet Works	(372)	none
.4	Power Intake Works	(9,863)	(13,374)
07.	Power Plant	13,216	13,964
.1	Powerhouse	(1,502)	(1,736)
.2	Turbines & Generators	(2,712)	(3,047)
.3	Accessory & Misc. Equip., Tailrace	(1,920)	(1,466)
.8	Transmission Plant	(7,082)	(7,715)
08.	Roads and Bridges	2,442	1,122
19.	Buildings, Grounds & Utilities	814	923
20.	Permanent Operating Equipment	827	580
30.	Engineering and Design	3,200	2,755
31.	Supervision and Administration	3,100	2,701
	TOTAL COST, FIRST STAGE DEVELOPMENT	40,300	41,500

Second Stage Development

Cost Acct No.	Feature	Present Estimate Oct 1965 Base	Latest Approved Estimate July 1965 Base
04.-	Dams	8,412	9,770
.4	Power Intake Works	(8,412)	(9,770)
07.	Power Plant	2,388	3,236
.1	Powerhouse	(95)	(472)
.2	Turbines and Generators	(1,555)	(1,706)
.3	Accessory & Misc. Equip., Tailrace	(249)	(420)
.8	Transmission Plant	(489)	(638)
30.	Engineering and Design	1,200	1,120
31.	Supervision and Administration	1,000	974
	TOTAL COST, SECOND STAGE DEVELOPMENT	13,000	15,100
	TOTAL PROJECT COST, FULL DEVELOPMENT	53,300	56,600

20.07 Explanation of Variations.- The principal items accounting for the $3,300,000 decrease in project costs indicated above are: (a) more detailed investigation and design has resulted in reduced diameters for power tunnels and penstocks; (b) the decision to utilize high-strength steel in the penstocks resulted in greatly reduced steel quantities; (c) more complete analysis of the installed costs of powerplant equipment; (d) changes in the transmission line route reduced the length of steel tower section; and (e) more detailed analysis of unit costs, based on latest available information.

20.08 Three additional features have been added to the project plan at the request of the Bureau of Reclamation. These features and the cost of each are as follows:

Reservoir outlet works	$ 559,000
Penstock interconnection	173,000
Transmission access roads	1,788,000
TOTAL	$2,520,000

20.09 The decision by the Bureau of Reclamation to forego construction of family housing at the site, and the provision of a single dormitory in its place, resulted in a reduction of approximately $400,000 in the cost of the project.

20.10 The principal factors accounting for the $525,000 increase in estimated engineering and design costs are: (a) the isolated location of the project, resulting in high transportation and camp support costs; (b) the rugged terrain, the dispersed arrangement of project features and the complete lack of roads and trails, requiring helicopter support for practically all field operations; (c) the heavy forest cover which hampers normal survey activities, and requires expensive clearing operations for even preliminary surveys; (d) although the cost of the project is not great in comparison with many other multiple-purpose projects, it presents an unusual number of factors requiring comparative studies of alternative plans; (e) the unusually high relative cost of powerhouse and powerplant equipment design, resulting from the fact that although the construction cost of the plant is not great it is as complex and requires the same degree of engineering effort as in the case of a much larger and more costly plant.

20.11 The engineering activities which have been performed to date have resulted in considerable reduction in project costs and in added project accomplishments. It is believed that additional savings can be effected by continued expansion of the engineering program. These activities constitute Value Engineering, and should be so considered in the evaluation of overall engineering and design costs.

20.12 The increase of $425,000 in the estimated cost of supervision and administration is principally accounted for by the unfavorable conditions of location and access discussed in paragraph 20.10 above. An unusually large staff of inspectors will be required to maintain continuity of observation, with contractors' operations in progress at many locations simultaneously. The provision of helicopter transportation to carry Government personnel to isolated feature sites, especially along the transmission line, will be an added item of cost. The supervision and administration estimate also includes project and district office costs, and therefore reflects the additional costs of these items.

SECTION 21 - PROJECT BENEFITS & ECONOMICS

21.01 General.- Benefits creditable to the Snettisham Project will be derived solely from sale of power in the Juneau area. Optimum development of the project includes construction of a dam at the Long Lake outlet which will raise the power pool elevation to 895 mean sea level. Further studies to be made prior to development of the Crater Lake phase of the project, may indicate the desirability of constructing a dam at that site also. However, this memorandum is essentially concerned with proper development of the Long Lake phase. The paragraphs following summarize and evaluate benefits, preliminary cost allocations and establish a benefit cost ratio for the project.

21.02 Power Benefits.- As previously mentioned, all benefits will be attributed directly to power revenue. Studies detailed in Appendix A show that Long Lake will have on-site dependable capacity of 46,700 kilowatts and will produce a gross of 225 million kilowatt-hours per year. The decision to make all generating units the same size results in provision of 23,350 kilowatts of on-site dependable capacity and a gross annual production of 106 million kilowatt-hours for the Crater Lake phase. To determine the at-market values, station use and transmission line losses were assumed to total 6 percent for both capacity and energy. No benefits were assigned for secondary energy in any of the computations.

21.03 Power Market.- All power or energy produced by the Snettisham project will be transmitted by 45 miles of line and submarine cable to the Juneau-Douglas market area. Although there are definite possibilities of future heavy industrial use, the present assumed use of the power will be primarily utility loads. Very minor industrial demand in the area is the only exception.

21.04 Project and System Operating Conditions.- The 50-year economical life of the project will span two general conditions of operation. The first will be the period of growth which spans approximately 13 years from 1970 to 1983. During this period the three units will be brought on line as needed with the project reaching full capability in 1983. The second period is the remaining 37 years during which time the project can be assumed to be operating at full capacity and in conjunction with other projects or power producing media. The project will be operated in such a manner as to make maximum use of the available water and the installed units. There is no other use for water at the present time nor is one visualized for the future. There are no water requirements for fish passage, supply for existing streams, irrigation or flood control.

21.05 Power Output Capabilities.-

a. Based on the project and system operating conditions, as outlined above, the only restrictions on power output will be machine capability and demand from the Juneau area in conjunction with the available water supply.

b. Consideration was given to the possibility of putting the Crater Lake phase in service prior to Long Lake; however, such a procedure proved to be uneconomical. Examination of Figure 19 shows that if Crater Lake were added first in 1970 then the Long Lake phase would be required in 1972, less than three years later. Such a sequence of development would require both projects to be in service in 1972. Conversely, the recommended schedule will allow a deferment of the Crater Lake phase until 1979. Total investment may therefore be delayed over 9 years by developing Long Lake first, as compared to under 3 years if Crater were developed first.

21.06 Monetary Evaluation.- The at-market power valuation used in the monetary evaluation of the above generation was 6.91 mills per kilowatt-hour for net primary energy and $65.11 per kilowatt-year for dependable capacity. These values were furnished by the Federal Power Commission in their letter of 25 February 1964, Exhibit 2, which states, "the value of hydroelectric power at Juneau based on cost of steam-electric power estimated in 1960 was $65.11 per kilowatt-year for dependable capacity plus 6.91 mills per kilowatt-hour for energy. The total unit value of power for a 70 percent load factor delivery, amounts to 17.53 mills per kilowatt-hour. This value was based on financing of a 49,500-kilowatt steam-electric plant with cost of money at 7 percent and with taxes of 6.77 percent of the investment. The cost for fuel was based on bunker oil at Juneau delivered for $3.20 per barrel." The present wholesale rate for all distributor received power in the Juneau area ranges from 17 to 27 mills per KWH which is greater than the above unit value.

21.07 Differences from Project Document.- The difference in average annual benefits from project document are:

	Project Document	Present Evaluation
Firm Energy	$3,709,000	$6,029,000
Secondary Energy	85,000	90,000
Total	$3,794,000	$6,119,000

The major contributions to the above 61 percent increase were:

(1) Different values for both energy and capacity.

(2) Increased energy production due to the higher level of Long Lake and slightly higher inflows.

(3) A different method of computing benefits.

(4) Other minor considerations included higher efficiencies, more exact regulation studies and reduction of waste flow.

21.08 Economic Analysis and Justification.-

a. Average annual costs developed in Appendix A are summarized as follows:

	Average Annual Costs	
	50-year life	100-year life
Long Lake Phase	$2,279,000	$1,966,000
Complete Project	$2,748,000	$2,347,000

b. Benefit computations as developed in Appendix A are summarized as follows:

	Annual Benefits
Long Lake Phase	$4,319,000
Crater Lake Phase	$2,117,000
Total Project	$6,026,000

c. Benefit/Cost Ratio - Benefit-to-cost comparisons for the recommended project plan as derived from the above values are summarized in the following tabulation:

Project	50-year life			100-year life		
Phase	Ann.Costs	Ann.Ben.	B/CRatio	Ann.Costs	Ann.Ben.	B/CRatio
Long Lake	2,279,000	4,319,000	1.90	1,966,000	4,319,000	2.20
Complete Project	2,748,000	6,026,000	2.19	2,347,000	6,026,000	2.57

SECTION 22 - DISCUSSION, CONCLUSIONS AND RECOMMENDATION

22.01 Discussion.- The project plan as described in this design memorandum is essentially the same as that recommended in Design Memorandum No. 3. Alternative studies and analyses have been made of several features of the project. Various methods of foundation treatment for the Long Lake dam were considered, with the conclusion that the use of steel prestressing tendons to assure stability of the foundation was the most favorable alternative. The feasibility of an underground powerhouse was extensively investigated. Economic studies indicated that a conventional surface powerhouse would result in the lowest overall cost for the power installation. Two alternative routes for portions of the transmission line were studied. It was concluded that the slightly longer coastal route would present fewer construction and maintenance problems than the previously recommended overland route, and that it would be significantly less costly. A number of minor revisions in the location and characteristics of specific features resulted from more detailed investigations and design analyses. It is proposed that the project be constructed in two stages. The second stage, consisting of the Crater Lake power intake works and completion of the power installation, will be deferred for a period dependent upon the growth of the market for project power.

22.02 Conclusions.- It is concluded that the project plan as described in this design memorandum, and as discussed above, represents the most favorable combination of alternatives which could be developed. It includes a concrete gravity dam at Long Lake, with a minimum of excavation, and with prestressing tendons as required to assure stability of the foundation; a conventional surface powerhouse with underground penstocks; and a coastal route for the transmission line.

22.03 Recommendation.- The District Engineer recommends that the first stage of the Snettisham project be approved for construction in general accordance with the plans and designs presented in this General Design Memorandum.

TABLES

TABLE 1

SNOWMELT COMPUTATIONS WITH PROBABLE MAXIMUM PRECIPITATION OCCURRING ON 1

Date	Solar Radiation	Mean Daily Temp	Dew Point Temp	Wind Velocity	T_a-32	T_d-32	Solar Radiation Melt		
	I_i	T_a	T_d	v	T'_a	T'_d	$0.00244I_i$	$0.22T'_a$	$0.78T'_d$
	Langley's Col 1	°F Col 2	°F Col 3	MPH Col 4	°F Col 5	°F Col 6	Inches Col 7	Inches Col 8	Inches Col 9
May 16	746	40	27	18	8	-	1.82	1.76	-
17	747	40	31	18	8	-	1.82	1.76	-
18	130	45	45	50	13	13	.32	2.86	10.14
19	130	42	42	39	10	10	.32	2.20	7.80
20	130	40	40	35	8	8	.32	1.76	6.24
21	753	41	36	18	9	4	1.84	1.98	3.12
22	755	39	36	18	7	4	1.84	1.54	3.12
23	756	43	36	18	11	4	1.84	2.42	3.12
24	757	43	36	18	11	4	1.85	2.42	3.12
25	759	48	36	18	16	4	1.85	3.52	3.12
26	760	46	37	18	14	5	1.85	3.08	3.90
27	762	45	40	18	13	8	1.86	2.86	6.24
28	763	46	39	18	14	7	1.86	3.08	5.46
29	750	46	38	35	14	6	1.83	3.08	4.68
30	700	46	33	25	14	1	1.71	3.08	.78
31	600	44	36	35	12	4	1.46	2.64	3.12

NOTES:
1. Column 1, Radiation was calculated using CRREL Research Report No. 160, "Daily Sums of G
2. Columns 2, 3 and 4 were taken from the report prepared by the Hydrometeorological Sectio
3. Columns 5 through 14 are obtained as indicated in each column heading.
4. Rain for 18, 19 and 20 May is 4.6, 5.5 and 14.3 inches, respectively.
5. On 15 May there is an average of 52 inches water equivalent over the basin.
6. Average basin elevation above 900 feet elevation, excluding lake area, is 3000 feet. Al
7. Generalized snowmelt equation for an open area is $M=.00244I_i + .0067v(.22T'_a + .78T'_d) + .$

18, 19 & 20 MAY

Col 8 + Col 9 Inches Col 10	0.0067v Inches Col 11	Convection Condensation Melt (Col 10) x (Col 11) Inches Col 12	Rain Melt $.007P_xT'_a$ Inches Col 13	Total Snowmelt Cols. 7+12+13 Inches/day Col 14
1.76	.12	.21		2.03
1.76	.12	.21		2.03
13.00	.34	4.42	.26	5.09
10.00	.26	2.60	.31	4.23
8.00	.23	1.84	.80	2.96
5.10	.12	.61		2.45
4.66	.12	.56		2.40
5.54	.12	.66		2.50
5.54	.12	.66		2.51
6.64	.12	.80		2.65
6.98	.12	.84		2.69
9.10	.12	1.09		2.93
8.54	.12	1.02		2.88
7.76	.23	1.78		3.61
3.86	.17	.66		2.37
5.76	.23	1.32		2.78

Global Radiation for Cloudless Skies".

on, U. S. Weather Bureau.

11 snowmelt calculations were made at this elevation.

$.007P_xT'_a$

TABLE 1

TABLE 2

LONG LAKE WATER QUALITY OBSERVATI

Sample Location	Date	Depth Ft.	Sediment PPM	Temp. °F.	SiO_2 PPM	Ca PPM	Mg PPM	Na PPM	K PPM	HCO_3 PPM	CO PP
Lower	26Mar65	Composite	6	32 surface 39 bottom	1.6	2.4	0.4	0.9	0.5	7	0
Middle	26Mar65	Composite	4	32 surface 39 bottom	1.2	2.0	0.2	1.4	0.5	6	0
Upper	26Mar65	Composite	4	32 surface 39 bottom	1.4	2.4	0.1	1.2	0.4	7	0
Lower	22Jul65	Composite	-	-	1.1	2.2	0.1	0.8	-	7	0
	"	5	2	42							
	"	50	4	42							
	"	100	4	42							
	"	150	4	42							
	"	200	3	41							
	"	250	3	41							
	"	300	4	41							
	"	350	4	41							
	"	400	4	40.5							
	"	450	3	40.5							
Middle	22Jul65	Composite	-	-	0.8	4.0	0.5	1.1	-	8	0
	"	5	5	41							
	"	50	4	41							
	"	100	4	40.5							
	"	150	6	40							
	"	200	4	40							
	"	250	5	40							
	"	300	5	40							
	"	350	4	40							
	"	400	5	40							
	"	450	4	40							
Upper	22Jul65	Composite	-	-	1.2	1.4	1.2	1.3	-	8	4
	"	5	67 1/	41							
	"	50	6	41							
	"	100	6	41							
	"	150	400 2/	41							
Inflow	22Jul65	Surface	10	41							

1/ Organic Material
2/ Bottom disturbed while sampling

ONS

O_3 PPM	SO_4 PPM	Cl PPM	Dissolved Solids PPM	Hardness $CaCO_3$ PPM	Hardness Non-Carb PPM	Specific Conductance micro mhos/cm at 25°C	pH	Color
[illegible]	1.0	3.5	13	8	2	21	6.7	5
[illegible]	1.0	2.8	12	6	1	17	6.8	5
[illegible]	1.0	3.2	13	6	0	20	6.9	5
[illegible]	1.9	2.8	15	6	0	21	6.9	5
						27		
						20		
						18		
						22		
						18		
						19		
						18		
						18		
						18		
						23		
[illegible]	1.4	2.8	19	6	0	21	6.9	5
						21		
						18		
						20		
						18		
						18		
						22		
						21		
						21		
						20		
						20		
[illegible]	1.9	4.3	23	7	1	27	7.0	5
						17		
						17		
						17		
						17		
						17		

TABLE 2

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SNETTISHAM PROJECT, ALASKA

TABLE 3 - SUMMARY COST ESTIMATE
Price Level - October 1965

Cost Acct No.		Item Cost in $1,000	Feature Cost in $1,000
	FIRST STAGE DEVELOPMENT		
01.	Lands and Damages		3
04.-	Dams		16,698
.1	Main Dam and Spillway	6,463	
.3	Outlet Works	372	
.4	Power Intake Works	9,863	
07.-	Power Plant		13,216
.1	Powerhouse	1,502	
.2	Turbines and Generators	2,712	
.3	Accessory & Misc. Equipt: Tailrace	1,920	
.31	Accessory Electrical Equipt.	(1,116)	
.32	Misc. Power Plant Equipt.	(567)	
.33	Tailrace	(237)	
.8-	Transmission Plant	7,082	
.80	Land & Land Rights	(42)	
.81	Clearing Land & Rights of Way	(441)	
.82	Structures and Improvements	(162)	
.83	Station Equipment	(1,149)	
.84	Towers & Fixtures	(367)	
.85	Poles & Fixtures	(1,029)	
.86	Overhead Conductors & Devices	(1,216)	
.88	Submarine Conductor & Devices	(888)	
.89	Roads and Trails	(1,788)	
08.	Roads and Bridges		2,442
19.	Buildings, Grounds and Equipment		814
20.	Permanent Operating Equipment		827
30.	Engineering and Design		3,200
31.	Supervision and Administration		3,100
	TOTAL COST, FIRST STAGE DEVELOPMENT		40,300
	SECOND STAGE DEVELOPMENT		
04.-	Dams		8,412
.4	Power Intake Works	8,412	
07.-	Power Plant		2,388
.1	Powerhouse	95	
.2	Turbines and Generators	1,555	
.31	Accessory Electrical Equipment	249	
.8	Transmission Plant	489	
30.	Engineering and Design		1,200
31.	Supervision and Administration		1,000
	TOTAL COST, SECOND STAGE DEVELOPMENT		13,000
	TOTAL PROJECT COST, FULL DEVELOPMENT		53,300

SNETTISHAM PROJECT, ALASKA

TABLE 4 - ALLOCATION OF DISTRIBUTIVE COSTS (in $1,000)

		DISTRIBUTION				
		FIRST STAGE				SECOND STAGE
Item	Item Cost	Main Dam and Spillway	Outlet Works	Power Intake Works	Power Plant	Power Intake Works
Diversion Works, Long Lake	404	354	21	29		
Diversion Works, Crater Lake	684					684
Temporary Field Facilities 1/	865	170	10	281	404 2/	
Temporary Access Roads	2,068	553	32	640	—	843
Subtotal	4,021	1,077	63	950	404	1,527
Direct Construction Cost		5,386	309	8,913	12,812	6,885
TOTAL CONSTRUCTION COST		6,463	372	9,863	13,216	8,412

1/ Distributed in proportion to feature cost.

2/ Applied to Powerhouse cost.

SNETTISHAM PROJECT, ALASKA

TABLE 5 - DETAILED COST ESTIMATE
Price Level - October 1965

FIRST STAGE DEVELOPMENT

Feature or Item	Unit	Quantity	Unit Price	Amount
LANDS AND DAMAGE				
TOTAL COST, LANDS AND DAMAGE				$ 3,000
DAMS				
Main Dam and Spillway				
Mobilization and Demobilization	Job	1		275,000
Excavation, Rock	CY	1,920	$ 5.50	10,600
Foundation Preparation	SY	5,520	8.25	45,500
Drilling Drain Holes	LF	12,350	9.90	122,300
Foundation Grouting	CF	9,000	6.60	59,400
Concrete				
Mass	CY	65,040	40.70	2,647,100
Structural	CY	130	88.00	11,400
Cement	Bbl	49,000	14.30	700,700
Reinforcing Steel	Lb	75,000	0.24	18,000
Miscellaneous Metal	Lb	20,000	1.05	21,000
Prestressing Tendons, Installed	LF	60,000	12.87	772,200
Subtotal				4,683,200
Contingencies 15%				702,800
Total Cost, Main Dam and Spillway				$ 5,386,000
Outlet Works				
Outlet Tunnel				
Excavation	CY	1,590	$ 55.00	$ 87,500
Concrete, Lining	CY	700	71.50	50,100
Cement	Bbl	1,050	14.30	15,000
Reinforcing Steel	Lb	152,400	0.24	36,600
Rock Bolts	Ea	480	49.50	23,800
Pressure Grouting	CF	80	6.60	500
Drilling Grout Holes	LF	420	6.60	2,800
Subtotal				216,300
Contingencies 15%				32,700
Total Cost, Outlet Tunnel				$ 249,000

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
DAMS, Continued				
Outlet Works (continued)				
Outlet Control Gate				
Excavation, Rock	CY	580	$ 2.75	$ 1,600
Concrete, Structural	CY	80	88.00	7,000
Cement	Bbl	120	14.30	1,700
Reinforcing Steel	Lb	4,850	0.24	1,200
Anchor Bars	LF	500	12.90	6,500
Gate, Gate Hoists and Frames	Lb	25,000	1.20	30,000
Subtotal				48,000
Contingencies 25%				12,000
Total Cost, Outlet Control Gate				$ 60,000
Total Cost, Outlet Works				$ 309,000
Power Intake Works				
Intake Structure				
Excavation, Lake Sediment	CY	15,800	$ 2.20	$ 34,800
Excavation, Rock	CY	1,200	6.60	7,900
Concrete, Structural	CY	1,380	110.00	151,800
Cement	Bbl	2,070	14.30	29,600
Reinforcing Steel	Lb	138,000	0.24	33,100
Trashrack	Lb	23,300	0.83	19,300
Power Tunnel Bulkhead, Complete	Lb	20,000	1.20	24,000
Outlet Tunnel Bulkhead, Complete	Lb	19,700	1.20	23,600
Outlet Tunnel Emergency Gate, Complete	Lb	36,400	1.20	43,700
Access Bridge				
Concrete	CY	30	110.00	3,300
Cement	Bbl	50	14.30	700
Reinforcing Steel	Lb	8,000	0.24	1,900
Subtotal				373,700
Contingencies 20%				74,300
Total Cost, Intake Structure				$ 448,000
Power Tunnel				
Excavation, Rock	CY	34,300	$ 55.00	$ 1,886,500
Concrete, Tunnel Lining	CY	15,000	71.50	1,072,500
Cement	Bbl	22,500	14.30	321,800
Reinforcing Steel	Lb	2,186,000	0.24	524,600
Structural Steel Tunnel Supports	Lb	79,000	0.46	36,300
Timber Lagging	Mbm	21	550.00	11,600
Rock Bolts	Ea	10,230	49.50	506,400

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
DAMS, continued				
Power Intake Works (continued)				
Power Tunnel (continued)				
Pressure Grouting in Tunnel	CF	2,500	$ 6.60	$ 16,500
Drilling Grout Holes	LF	5,000	6.60	33,000
Subtotal				4,409,200
Contingencies 15%				661,800
Total Cost, Power Tunnel				$ 5,071,000
Surge Tank				
Excavation, Rock, Open Cut	CY	4,050	$ 16.50	$ 66,800
Excavation, Shaft	CY	5,680	77.00	437,400
Concrete, Lining	CY	1,320	88.00	116,200
Cement	Bbl	1,970	14.30	28,200
Reinforcing Steel	Lb	375,300	0.24	90,100
Steel, Riser Lining (A-516)	Lb	12,700	0.66	8,400
Subtotal				747,100
Contingencies 15%				111,900
Total Cost, Surge Tank				$ 859,000
Valve Vault and Access Adit				
Excavation				
Surface	CY	180	$ 6.60	$ 1,200
Adit	CY	1,440	55.00	79,200
Vault	CY	730	71.50	52,200
Concrete, Lining	CY	140	71.50	10,000
Concrete, Paving	CY	110	55.00	6,100
Cement	Bbl	370	14.30	5,300
Reinforcing Steel	Lb	13,800	0.24	3,300
Rock Bolts	Ea	490	49.50	24,300
Chain Link Fabric	SF	14,340	0.99	14,200
Subtotal				195,800
Contingencies 15%				29,200
Total Cost, Valve Vault and Access Adit				$ 225,000
Penstock				
Tunnel Excavation	CY	7,260	$ 55.00	$ 399,300
Concrete, Backfill in Tunnel	CY	4,790	71.50	342,500
Cement	Bbl	7,180	14.30	102,700
Steel, Penstock Liner (A-516)	Lb	160,000	0.66	105,600
Steel, Penstock Liner (A-517)	Lb	690,100	0.89	614,200
Steel, Tunnel Supports	Lb	23,300	0.46	10,700
Timber Lagging	Mbm	7	550.00	3,900

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
DAMS, Continued				
Power Intake Works (continued)				
Penstock (continued)				
Wye Branches (A-517)	Lb	25,300	$ 0.94	$ 23,800
Butterfly Valve, Complete	Lb	100,000	1.93	193,000
Rock Bolts	Ea	2,190	49.50	108,400
Subtotal				1,904,100
Contingencies 15%				285,900
Total Cost, Penstock				$ 2,190,000
Penstock, Crater Lake, First Stage				
Tunnel Excavation	CY	690	$ 55.00	$ 38,000
Concrete, Backfill in Tunnel	CY	140	71.50	10,000
Cement	Bbl	210	14.30	3,000
Steel, Penstock Liner (A-517)	Lb	20,400	0.89	18,200
Steel, Tunnel Supports	Lb	2,600	0.46	1,200
Timber Lagging	Mbm	1	550.00	600
Rock Bolts	Ea	250	49.50	12,400
Wye Branch (A-517)	Lb	22,000	0.94	20,700
Subtotal				104,100
Contingencies 15%				15,900
Total Cost, Penstock, Crater Lake, First Stage				$ 120,000
Total Cost, Power Intake Works				$ 8,913,000
TOTAL COST, DAMS				$14,608,000
POWER PLANT				
Powerhouse				
Excavation, Rock	CY	9,050	$ 6.60	$ 59,700
Excavation, Common	CY	50	2.20	100
Backfill	CY	500	1.10	600
Concrete, Superstructure	CY	2,300	110.00	253,000
Concrete, Substructure	CY	2,450	30.80	75,500
Cement	Bbl	4,650	14.30	66,500
Reinforcing Steel	Lb	536,000	0.24	128,600
Doors	LS			4,400
Walls, Ceilings & Floors	LS			6,300
Painting	LS			33,900
Plumbing Fixtures	LS			2,100
Piping Systems	LS			211,700
Heating & Ventilating	LS			45,600

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
POWER PLANT, continued				
Powerhouse (continued)				
Miscellaneous Metals	LS			$ 6,000
Open-Web Roof Joists	Ea	24	$330.00	7,900
Steel Roof Deck	SF	7,250	0.24	1,700
Roofing	LS			11,700
Subtotal				915,300
Contingencies 20%				182,700
Total Cost, Powerhouse				$ 1,098,000
Turbines and Generators				
Turbines	Ea	2	564,500	$ 1,129,000
Generators	Ea	2	296,700	593,400
Governors	Ea	2	55,650	111,300
Spherical Valves	Ea	2	121,450	242,900
Pressure Relief Valves	Ea	2	91,650	183,300
Subtotal				2,259,900
Contingencies 20%				452,100
Total Cost, Turbines and Generators				$ 2,712,000
Accessory & Misc. Equipment; Tailrace				
Accessory Electrical Equipment				
15 KV 480V & Control Equipment	LS			$ 452,200
Misc. Electrical Equipment	LS			477,800
Subtotal				930,000
Contingencies 20%				186,000
Total Cost, Accessory Electrical Equipment				$ 1,116,000
Miscellaneous Power Plant Equipment				
75-Ton Bridge Crane	LS			$ 108,300
48" Dresser Couplings	Ea	4	$990.00	4,000
Draft Tube Stoplog Lifting Beam	LS			700
3-Ton Draft Tube Monorail	LS			7,700
Draft Tube Gates	LS			17,000
Draft Tube Gate Guides	LS			42,400
Fans	LS			73,200
Misc. Mechanical Equipment	LS			219,500
Subtotal				472,800
Contingencies 20%				94,200
Total Cost, Miscellaneous Power Plant Equipment				$ 567,000

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
POWER PLANT, Continued				
Accessory & Misc. Equipment; Tailrace (continued)				
Tailrace				
Unclassified Excavation	CY	31,800	$ 3.85	$ 122,400
Rockfill from Req'd Excavation	CY	2,400	2.48	6,000
Gravel Base Course	CY	220	6.60	1,500
Riprap	CY	400	8.80	3,500
Concrete Lining	CY	220	55.00	12,100
Concrete Sill	CY	40	55.00	2,200
Cement	Bbl	390	14.30	5,600
Steel Sheet Piling	LF	2,700	16.50	44,600
Subtotal				197,900
Contingencies 20%				39,100
Total Cost, Tailrace				$ 237,000
Total Cost, Accessory & Misc. Equipment; Tailrace				$ 1,920,000
Transmission Plant				
Lands & Land Rights				$ 33,600
Contingencies 25%				8,400
Total Cost, Lands & Land Rights				$ 42,000
Clearing Land & Rights of Way	LS			$ 353,100
Contingencies 25%				87,900
Total Cost, Clearing Land & Rights of Way				$ 441,000
Structures & Improvements	LS			$ 129,800
Contingencies 25%				32,200
Total Cost, Structures & Improvements				$ 162,000
Station Equipment				
Conduit	LS			$ 35,300
Control Equipment	LS			143,000
Conversion Equipment	LS			232,700
Fences	LS			21,800
Foundations and Settings	LS			7,500
General Station Equipment	LS			9,700
Primary & Secondary Voltage Connections	LS			232,800

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
POWER PLANT, Continued				
Transmission Plant (continued)				
Station Equipment (continued)				
Switching Equipment	LS			$ 231,400
Tools & Appliances	LS			4,600
Subtotal				918,800
Contingencies 25%				230,200
Total Cost, Station Equipment				$ 1,149,000
Towers and Fixtures				
Foundation Excavation and Backfill	LS			$ 49,000
Towers	LS			244,200
Subtotal				293,200
Contingencies 25%				73,800
Total Cost, Towers and Fixtures				$ 367,000
Poles and Fixtures				
Anchors	LS			$ 213,500
Brackets	LS			68,800
Crossarms and Braces	LS			137,000
Excavation and Backfill	LS			189,600
Poles	LS			158,100
Settings	LS			56,500
Subtotal				823,500
Contingencies 25%				205,500
Total Cost, Poles and Fixtures				$ 1,029,000
Overhead Conductors and Devices				
Conductors	LS			$ 697,400
Ground Wires and Clamps	LS			4,400
Insulators	LS			204,000
Other Devices	LS			66,800
Subtotal				972,600
Contingencies 25%				243,400
Total Cost, Overhead Conductors and Devices				$ 1,216,000
Submarine Conductors and Devices	LS			$ 710,600
Contingencies 25%				177,400
Total Cost, Submarine Conductors and Devices				$ 888,000

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
POWER PLANT, Continued				
Transmission Plant (continued)				
Roads and Trails				$ 1,430,000
Contingencies 25%				358,000
Total Cost, Roads and Trails				$ 1,788,000
Total Cost, Transmission Plant				$ 7,082,000
TOTAL COST, POWER PLANT				$12,812,000
ROADS AND BRIDGES				
Docking Facilities				
Dock and Ramp Structure	LS			$ 245,000
Excavation and Fill	LS			70,000
Dredging	CY	250,000	$ 1.05	260,000
Floats	LS			15,000
Subtotal				590,000
Contingencies 20%				118,000
Total Cost, Docking Facilities				$ 708,000
Access Road				
Clearing and Grubbing	LS			$ 55,000
Rock Excavation	LS			255,000
Earthwork	LS			550,000
Surfacing	LS			60,000
Riprap	LS			55,000
Crater Creek Bridge	LS			95,000
Glacier Creek Bridge	LS			50,000
Tailrace Channel Bridge	LS			175,000
Culverts	LS			150,000
Subtotal				1,445,000
Contingencies 20%				289,000
Total Cost, Access Road				$ 1,734,000
TOTAL COST, ROADS AND BRIDGES				$ 2,442,000

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
BUILDINGS, GROUNDS AND UTILITIES				
Site Preparation, Streets and Utilities				
Clearing	Ac	2	$1,650	$ 3,300
Excavation, Rock	CY	1,020	6.33	6,500
Fill, Rock (from excavation)	CY	17,620	3.03	53,400
Fill, Unclassified (from borrow)	CY	40,400	1.76	71,100
Streets and Utilities	LS			93,500
Subtotal				227,800
Contingencies 20%				45,200
Total Cost, Site Preparation, Streets and Utilities				$ 273,000
Housing and Service Buildings				
Dormitory	LS			$ 192,500
Powerhouse Office & Shop Bldg	LS			154,000
Transmission Maintenance Bldg	LS			84,700
Underground Passageway	LS			19,800
Subtotal				451,000
Contingencies 20%				90,000
Total Cost, Housing and Service Buildings				$ 541,000
TOTAL COST, BUILDINGS, GROUNDS AND UTILITIES				$ 814,000
PERMANENT OPERATING EQUIPMENT				
Transportation Equipment				
50-foot Personnel and Supply Boat	Ea	1		$ 160,000
60-foot Landing Craft	Ea	1		110,000
Station Wagon	Ea	1		4,000
Pick-Up, 1/2-ton 4WD	Ea	1		4,400
Pick-Up, 3/4-ton 4WD	Ea	1		5,000
Tracked Snow Vehicles	Ea	2	8,800	17,600
Subtotal				301,000
Contingencies 20%				60,000
Total Cost, Transportation Equipment				$ 361,000
Communications Equipment				
Two-way Radio System	LS			$ 5,500
Interconnecting Telephone System	LS			5,500
Carrier-Current Communications System	LS			165,000
Subtotal				176,000
Contingencies 20%				35,000
Total Cost, Communications Equipment				$ 211,000

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
PERMANENT OPERATING EQUIPMENT, Continued				
Miscellaneous Operating Equipment				
Line Truck	Ea	1		$ 13,900
Tractor	Ea	1		30,800
Road Grader-Snow Plow	Ea	1		29,700
Powerhouse Office Equipment	LS			22,000
Maintenance & Machine Shop Tools and Equipment	LS			83,500
Warehouse Equipment	LS			16,500
Fire Equipment	LS			16,500
Subtotal				212,900
Contingencies 20%				42,100
Total Cost, Miscellaneous Operating Equipment				$ 255,000
TOTAL COST, PERMANENT OPERATING EQUIPMENT				$ 827,000
CONSTRUCTION FACILITIES				
Diversion Tunnel				
Excavation, Rock	CY	2,700	$ 55.00	$ 148,500
Rock Bolts	Ea	770	47.30	36,400
Concrete, Tunnel Lining	CY	180	71.50	12,900
Concrete Plug	CY	130	55.00	7,200
Cement	Bbl	470	14.30	6,700
Reinforcing Steel	Lb	48,000	0.24	11,500
Pneumatic Mortar Lining	SF	6,500	3.85	25,000
Welded Wire Mesh	SF	6,500	0.57	3,700
Blasting Rock Plug	Job	1		71,500
Subtotal				$ 323,400
Contingencies 25%				80,600
Total Cost, Diversion Tunnel				$ 404,000
Temporary Field Facilities				
Government Camp	LS			$ 275,000
Site Work, Contractor's Camp	LS			137,500
Utilities, Contractor's Camp	LS			88,000
Field Office and Laboratory	LS			110,000
Communication Facilities	LS			110,000
Subtotal				720,500
Contingencies 20%				144,500
Total Cost, Temporary Field Facilities				$ 865,000

FIRST STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
CONSTRUCTION FACILITIES, Continued				
Temporary Access Roads				
Access Road, Long Lake	LF	5,100	$ 99.00	$ 505,000
Access Road, Surge Tank	LF	4,800	99.00	475,000
Subtotal				980,000
Contingencies 25%				245,000
Total Cost, Temporary Access Roads				$ 1,225,000
TOTAL COST, CONSTRUCTION FACILITIES				$ 2,494,000
TOTAL CONSTRUCTION COST				$34,000,000
ENGINEERING AND DESIGN				$ 3,200,000
SUPERVISION AND ADMINISTRATION				$ 3,100,000
TOTAL PROJECT COST (First Stage Development Only)				$40,300,000
SECOND STAGE DEVELOPMENT				
DAMS				
Power Intake Works				
Intake Structure				
Excavation, Service Area	CY	5,910	$ 6.60	$ 39,000
Excavation, Intake Structure	CY	2,340	6.60	15,400
Concrete, Structural	CY	302	110.00	33,200
Cement	Bbl	403	14.30	5,800
Reinforcing Steel	Lb	20,000	0.24	4,800
Anchor Bars, Drill & Grout	LF	1,900	12.87	24,500
Drill 30" Diameter Shaft	LF	210	143.00	30,000
Steel Shaft Liner & Vent	Lb	6,800	1.21	8,200
Grout Shaft Liner	CF	380	6.60	2,500
Trashrack, Hoist and Lifting Beam	Lb	4,000	1.21	4,800
Intake Gate, Frame, Guides and Hoist	Lb	97,500	1.21	118,000
Subtotal				286,200
Contingencies 25%				69,800
Total Cost, Intake Structure				$ 356,000
Power Tunnel				
Tunnel Excavation	CY	17,460	$ 55.00	$ 960,000
Concrete, Tunnel Lining	CY	11,200	71.50	800,800
Cement	Bbl	16,800	14.30	240,200
Reinforcing Steel	Lb	1,632,200	0.24	391,700

SECOND STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
DAMS, Continued				
Power Intake Works (continued)				
Power Tunnel (continued)				
Rock Bolts	Ea	7,200	$ 49.50	$ 356,400
Steel Tunnel Supports	Lb	66,600	0.46	30,600
Timber Lagging	Mbm	20	550.00	11,000
Pressure Grouting in Tunnel	CF	2,000	6.60	13,200
Drilling Grout Holes	LF	4,000	6.60	26,400
Subtotal				2,830,300
Contingencies 25%				687,700
Total Cost, Power Tunnel				$ 3,518,000
Surge Tank				
Rock Excavation, Open Cut	CY	4,050	$ 16.50	$ 66,800
Rock Excavation, Shaft	CY	5,680	77.00	437,400
Concrete, Shaft Liner	CY	1,320	88.00	116,200
Cement	Bbl	1,970	14.30	28,200
Reinforcing Steel	Lb	375,300	0.24	90,100
Steel, Riser Lining (A-516)	Lb	12,700	0.66	8,400
Subtotal				747,100
Contingencies 25%				181,900
Total Cost, Surge Tank				$ 929,000
Valve Vault and Access Adit				
Excavation				
Surface	CY	180	$ 6.60	$ 1,200
Adit	CY	1,440	55.00	79,200
Vault	CY	730	71.50	52,200
Concrete, Lining	CY	140	71.50	10,000
Concrete, Paving	CY	110	55.00	6,100
Cement	Bbl	370	14.50	5,300
Reinforcing Steel	Lb	13,800	0.24	3,300
Rock Bolts	Ea	490	49.50	24,300
Chain Link Fabric	SF	14,340	0.99	14,200
Subtotal				195,800
Contingencies 25%				48,200
Total Cost, Valve Vault and Access Adit				$ 244,000
Penstock				
Tunnel Excavation	CY	5,250	$ 55.00	$ 288,800
Concrete, Backfill in Tunnel	CY	4,030	71.50	288,100
Cement	Bbl	6,040	14.30	86,400
Steel, Penstock Liner (A-516)	Lb	89,000	0.66	58,700

SECOND STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
DAMS, Continued				
Power Intake Works (continued)				
Penstock (continued)				
Steel, Penstock Liner (A-517)	Lb	610,000	$ 0.89	$ 542,900
Steel Tunnel Supports	Lb	15,540	0.46	7,100
Timber Lagging	Mbm	4	550.00	2,200
Rock Bolts	Ea	1,790	49.50	88,600
Butterfly Valve, Complete	Lb	60,000	1.93	115,800
Subtotal				1,478,600
Contingencies 25%				359,400
Total Cost, Penstock				$ 1,838,000
Total Cost, Power Intake Works				$ 6,885,000
TOTAL COST, DAMS				$ 6,885,000
POWER PLANT				
Powerhouse				
Concrete Substructure	CY	690	$ 30.80	$ 21,300
Cement	Bbl	1,040	14.30	14,900
Piping System	LS			33,100
Heating & Ventilating	LS			7,300
Subtotal				76,600
Contingencies 25%				18,400
Total Cost, Powerhouse				$ 95,000
Turbines and Generators				
Turbine	Ea	1		$ 564,500
Generator	Ea	1		296,700
Governor	Ea	1		55,700
Spherical Valves	Ea	2	121,450	242,900
Pressure Relief Valve	Ea	1		91,700
Subtotal				$ 1,251,500
Contingencies 25%				303,500
Total Cost, Turbines and Generators				$ 1,555,000
Accessory Electrical Equipment				
15 KV, 480V & Control Equipment	LS			$ 102,100
Miscellaneous Electrical Equip.	LS			97,800
Subtotal				199,900
Contingencies 25%				49,100
Total Cost, Accessory Electrical Equipment				$ 249,000

SECOND STAGE DEVELOPMENT, Continued

Feature or Item	Unit	Quantity	Unit Price	Amount
POWER PLANT, Continued				
Transmission Plant	LS			$ 393,800
Contingencies 25%				95,200
Total Cost, Transmission Plant				$ 489,000
TOTAL COST, POWER PLANT				$ 2,388,000
CONSTRUCTION FACILITIES				
Diversion Tunnel				
Portal Excavation	CY	230	$ 2.75	$ 600
Tunnel Excavation	CY	5,180	55.00	284,900
Shaft Excavation	CY	1,900	41.53	78,900
Concrete, Structural	CY	250	88.00	22,000
Concrete Plug	CY	170	55.00	9,400
Cement	Bbl	630	14.30	9,000
Reinforcing Steel	Lb	19,100	0.24	4,600
Gate and Gate Guides	Lb	8,000	1.21	9,700
Rock Bolts	Ea	1,200	49.50	59,400
Blasting Rock Plug	Job	1		71,500
Subtotal				550,000
Contingencies 25%				134,000
Total Cost, Diversion Tunnel				$ 684,000
Temporary Access Roads				
Access Road, Surge Tank	LF	550	$ 99.00	$ 54,500
Access Road, Crater Lake	LF	6,300	99.00	623,700
Subtotal				678,200
Contingencies 25%				164,800
Total Cost, Temporary Access Roads				$ 843,000
TOTAL COST, CONSTRUCTION FACILITIES				$ 1,527,000
TOTAL CONSTRUCTION COST				$10,800,000
ENGINEERING AND DESIGN				$ 1,200,000
SUPERVISION AND ADMINISTRATION				$ 1,000,000
TOTAL PROJECT COST (Second Stage Development Only)				$13,000,000
TOTAL PROJECT COST (Full Development)				$53,300,000

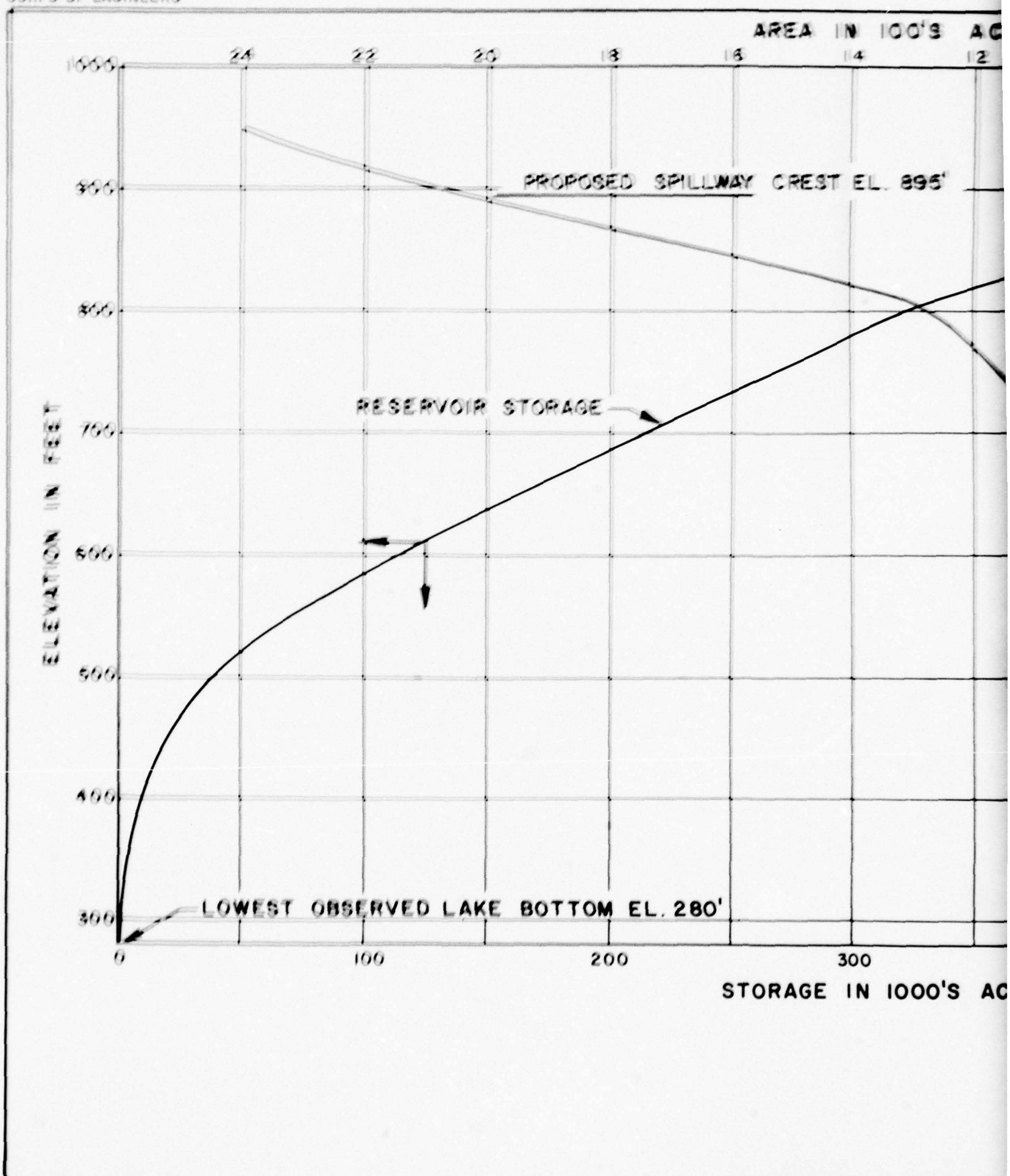
SNETTISHAM PROJECT, ALASKA

TABLE 6 - COST ESTIMATES, ALTERNATIVE DAM PLANS

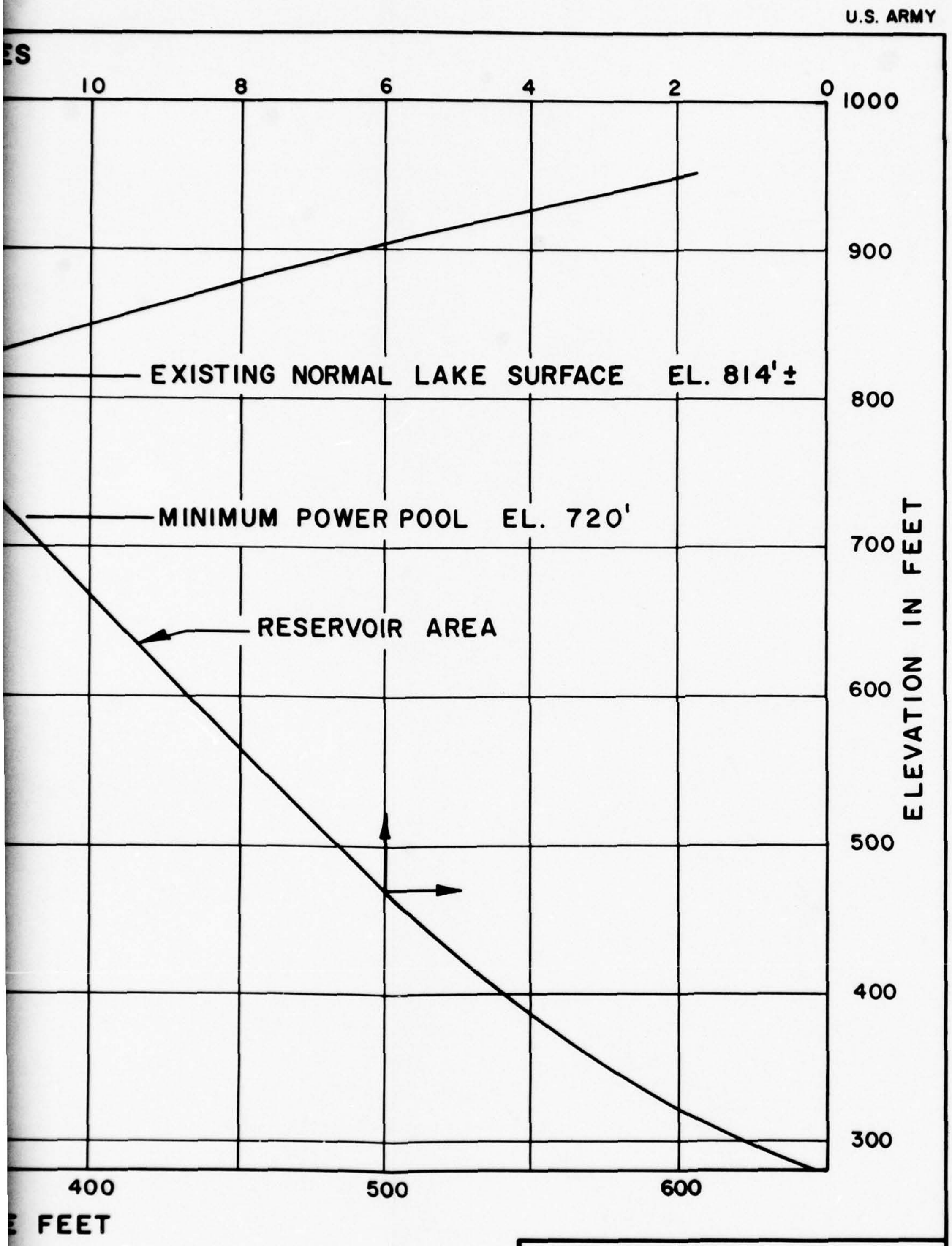
Price Level - October 1965

Feature or Item	Unit	Unit Price	PLAN A Quantity	PLAN A Amount	PLAN B Quantity	PLAN B Amount	PLAN C Quantity	PLAN C Amount
Mobilization and Demob.	Job		1	$ 275,000	1	$ 275,000	1	$ 275,000
Excavation, Rock								
Plan A & B	CY	5.50	1,920	10,600	1,920	10,600		
Plan C	CY	3.85					59,670	229,700
Foundation Preparation	SY	8.25	5,520	45,500	5,520	45,500	7,450	61,500
Drilling Grout Holes	LF	6.60	6,730	44,400	None		6,730	44,400
Drilling Drain Holes	LF	9.90	9,750	96,500	12,350	122,300	10,500	104,000
Foundation Grouting	CF	6.60	550	3,600	9,000	59,400	550	3,600
Concrete, Structural	CY	88.00	130	11,400	130	11,400	130	11,400
Concrete, Mass, Plan A & B	CY	40.70	65,040	2,647,100	65,040	2,647,100		
Concrete, Mass, Plan C	CY	30.80					104,430	3,216,400
Cement	Bbl	14.30	49,000	700,700	49,000	700,700	78,420	1,121,400
Reinforcing Steel	Lb	0.24	75,000	18,000	75,000	18,000	80,000	19,200
Miscellaneous Metal	Lb	1.05	20,000	21,000	20,000	21,000	20,000	21,000
Prestressing Tendons, Installed	LF	12.87	None		60,000	772,200	None	
Subtotal				3,873,800		4,683,200		5,107,600
Contingencies				581,200		702,800		766,400
TOTAL COST DAM				$4,455,000		$5,386,000		$5,874,000

FIGURES



AREA IN 100'S AC
24
22
20
18
16
14
12
1000
900
800
700
600
500
400
300
PROPOSED SPILLWAY CREST EL. 895'
RESERVOIR STORAGE
ELEVATION IN FEET
LOWEST OBSERVED LAKE BOTTOM EL. 280'
0
100
200
300
STORAGE IN 1000'S AC



SNETTISHAM PROJECT
FIRST STAGE DEVELOPMENT

LONG LAKE RESERVOIR
AREA STORAGE CURVES

U.S. ARMY ENGINEER DISTRICT, ALASKA
Planning and Reports Branch
Prepared by GNM Date 30 SEP 65

DESIGN MEMORANDUM NO.7 FIGURE I

2

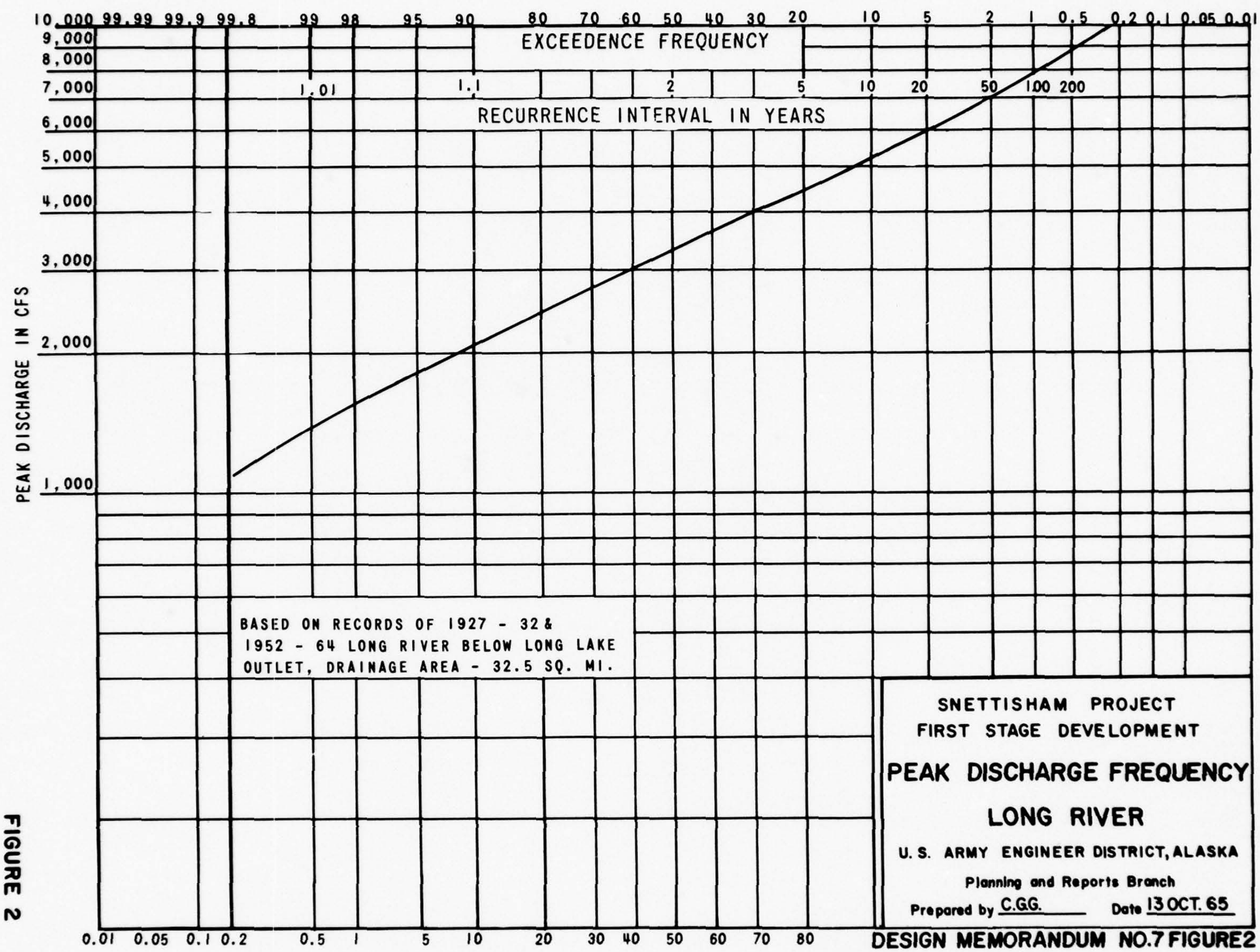
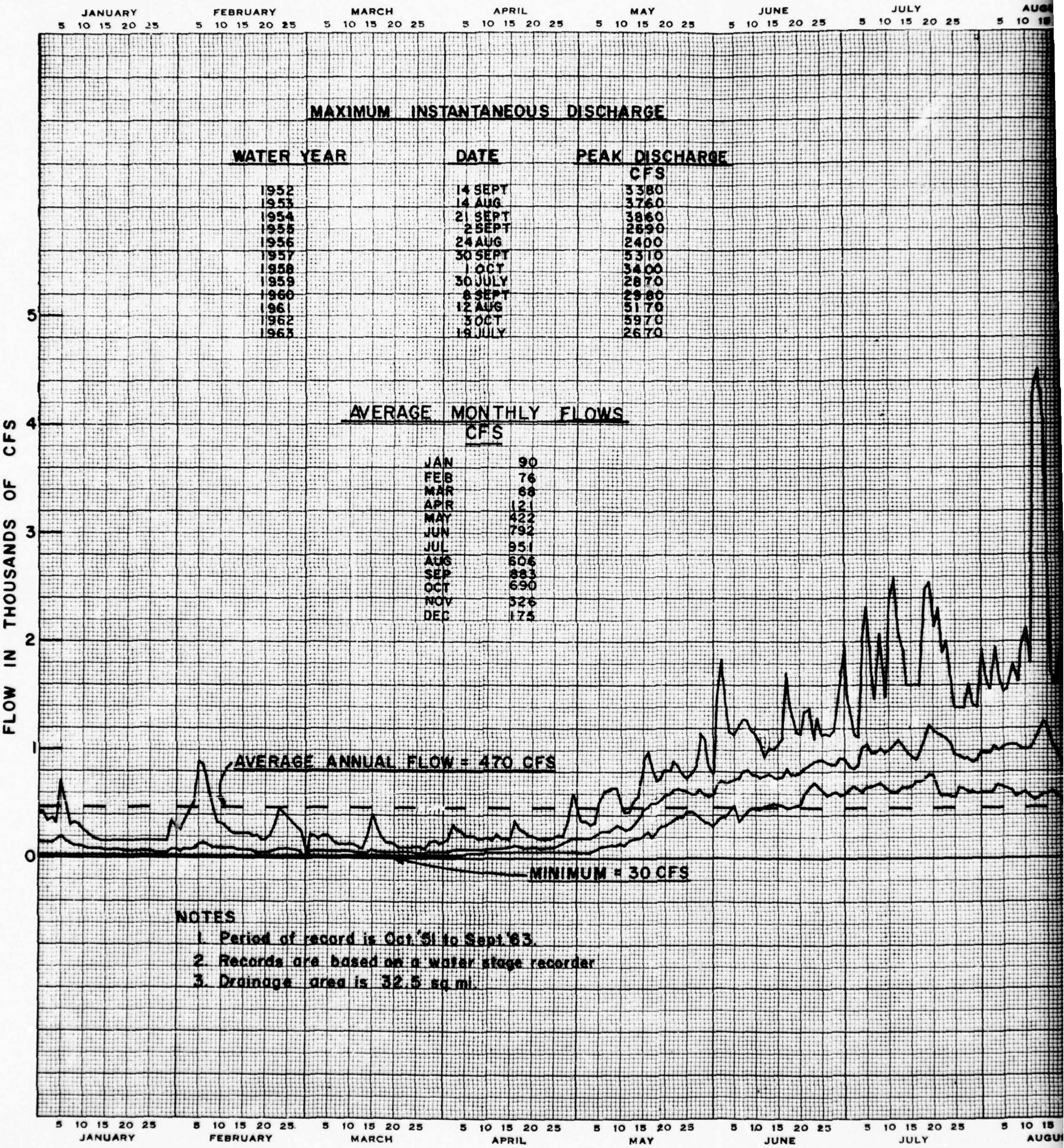

EXCEEDENCE FREQUENCY
RECURRENCE INTERVAL IN YEARS
PEAK DISCHARGE IN CFS
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
99.99 99.9 99.8 99 98 95 90 80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1 0.05 0.01
1.01 1.1 2 5 10 20 50 100 200
0.01 0.05 0.1 0.2 0.5 1 5 10 20 30 40 50 60 70 80
BASED ON RECORDS OF 1927 - 32 &
1952 - 64 LONG RIVER BELOW LONG LAKE
OUTLET, DRAINAGE AREA - 32.5 SQ. MI.
SNETTISHAM PROJECT
FIRST STAGE DEVELOPMENT
PEAK DISCHARGE FREQUENCY
LONG RIVER
U. S. ARMY ENGINEER DISTRICT, ALASKA
Planning and Reports Branch
Prepared by C.G.G. Date 13 OCT. 65
DESIGN MEMORANDUM NO.7 FIGURE2

FIGURE 2



MAXIMUM INSTANTANEOUS DISCHARGE

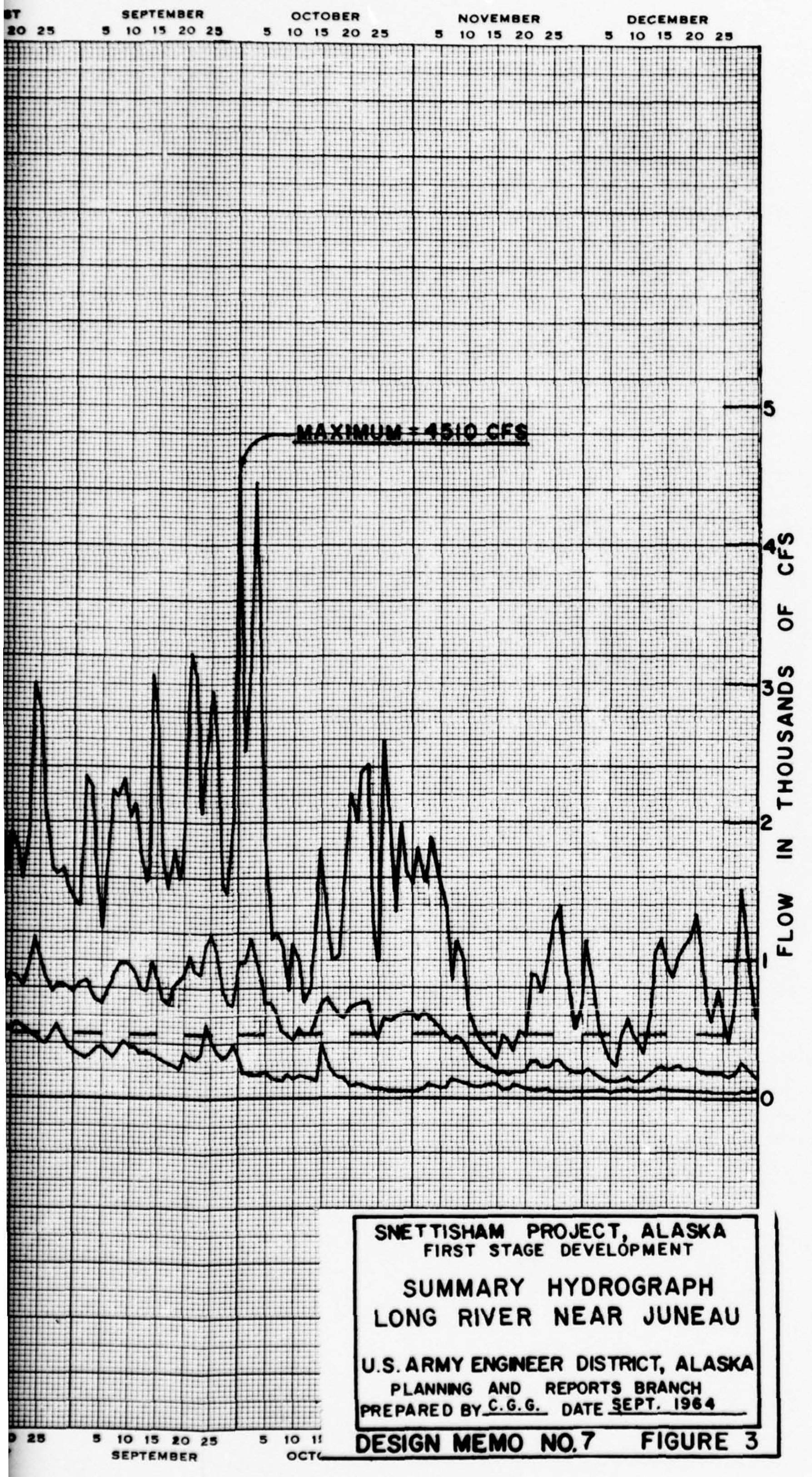
WATER YEAR	DATE	PEAK DISCHARGE CFS
1952	14 SEPT	3380
1953	14 AUG	3760
1954	21 SEPT	3860
1955	2 SEPT	2690
1956	24 AUG	2400
1957	30 SEPT	5310
1958	1 OCT	3400
1959	30 JULY	2870
1960	8 SEPT	2980
1961	12 AUG	5170
1962	3 OCT	5970
1963	19 JULY	2670

AVERAGE MONTHLY FLOWS CFS

JAN	90
FEB	76
MAR	68
APR	121
MAY	422
JUN	792
JUL	951
AUG	606
SEP	883
OCT	690
NOV	326
DEC	175

NOTES

1. Period of record is Oct.'51 to Sept.'63.
2. Records are based on a water stage recorder
3. Drainage area is 32.5 sq mi.

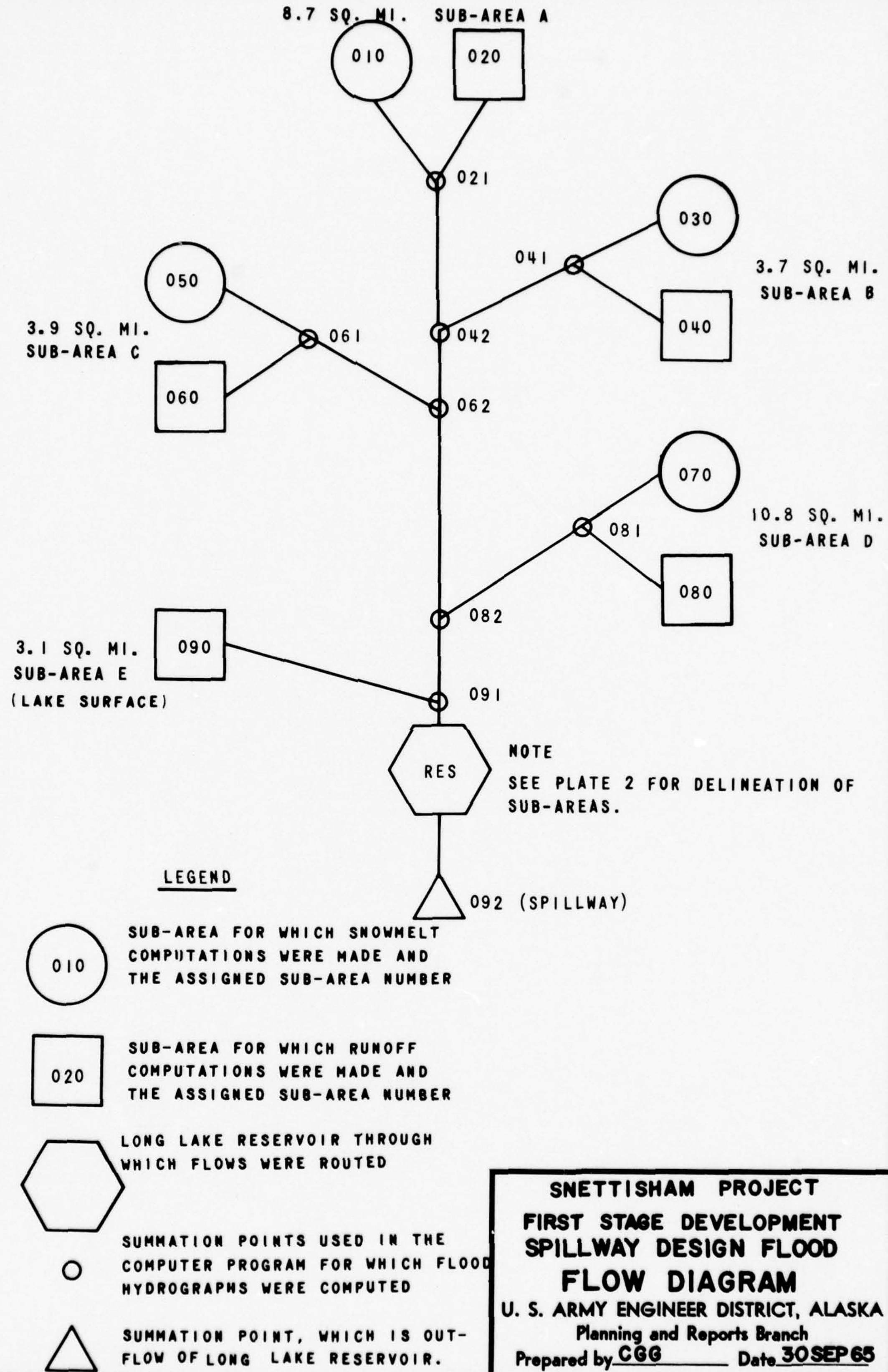


SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

SUMMARY HYDROGRAPH
LONG RIVER NEAR JUNEAU

U.S. ARMY ENGINEER DISTRICT, ALASKA
PLANNING AND REPORTS BRANCH
PREPARED BY C.G.G. DATE SEPT. 1964

DESIGN MEMO NO. 7 FIGURE 3



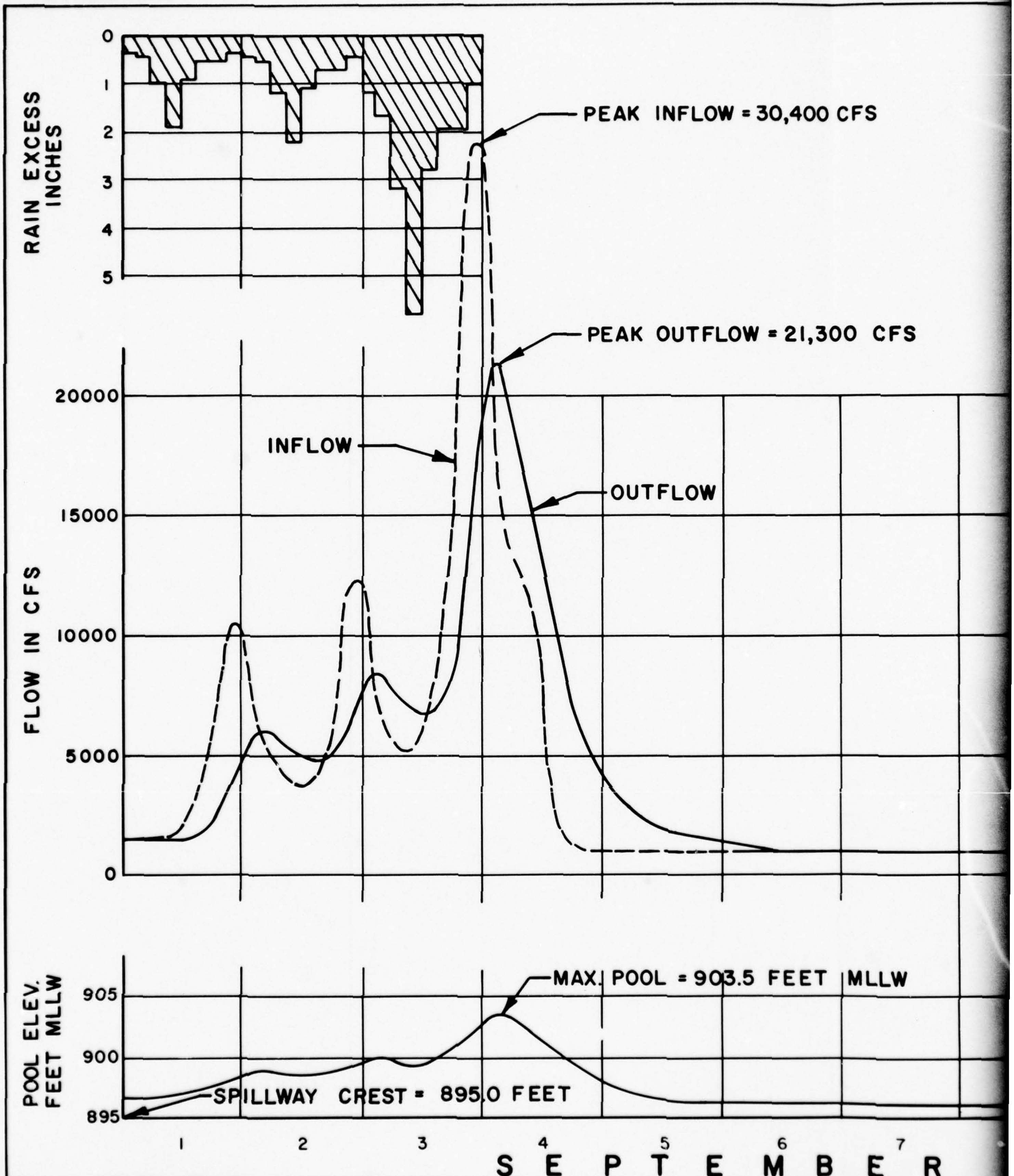
SNETTISHAM PROJECT

FIRST STAGE DEVELOPMENT
SPILLWAY DESIGN FLOOD

FLOW DIAGRAM

U. S. ARMY ENGINEER DISTRICT, ALASKA
Planning and Reports Branch
Prepared by CGG Date 30 SEP 65

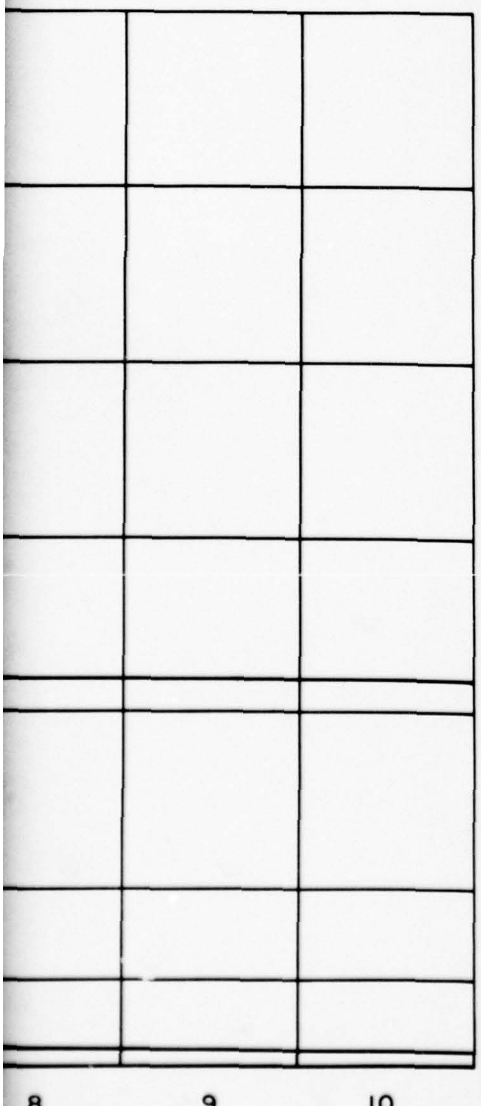
CORPS OF ENGINEERS



U.S. ARMY

NOTES:

1. INITIAL BASE FLOW IS 1400 cfs AND FINAL BASE FLOW IS 1000 cfs.
2. INITIAL POOL ELEVATION IS 896.3 FEET.
3. THREE DAY PROBABLE MAXIMUM PRECIPITATION IS 33.2 INCHES AND IS GIVEN IN 3 HOURS INCREMENTS ON THE HYETOGRAPH
4. LOSSES ARE CONSIDERED NEGLIGABLE.



SNETTISHAM PROJECT
FIRST STAGE DEVELOPMENT

SEPTEMBER

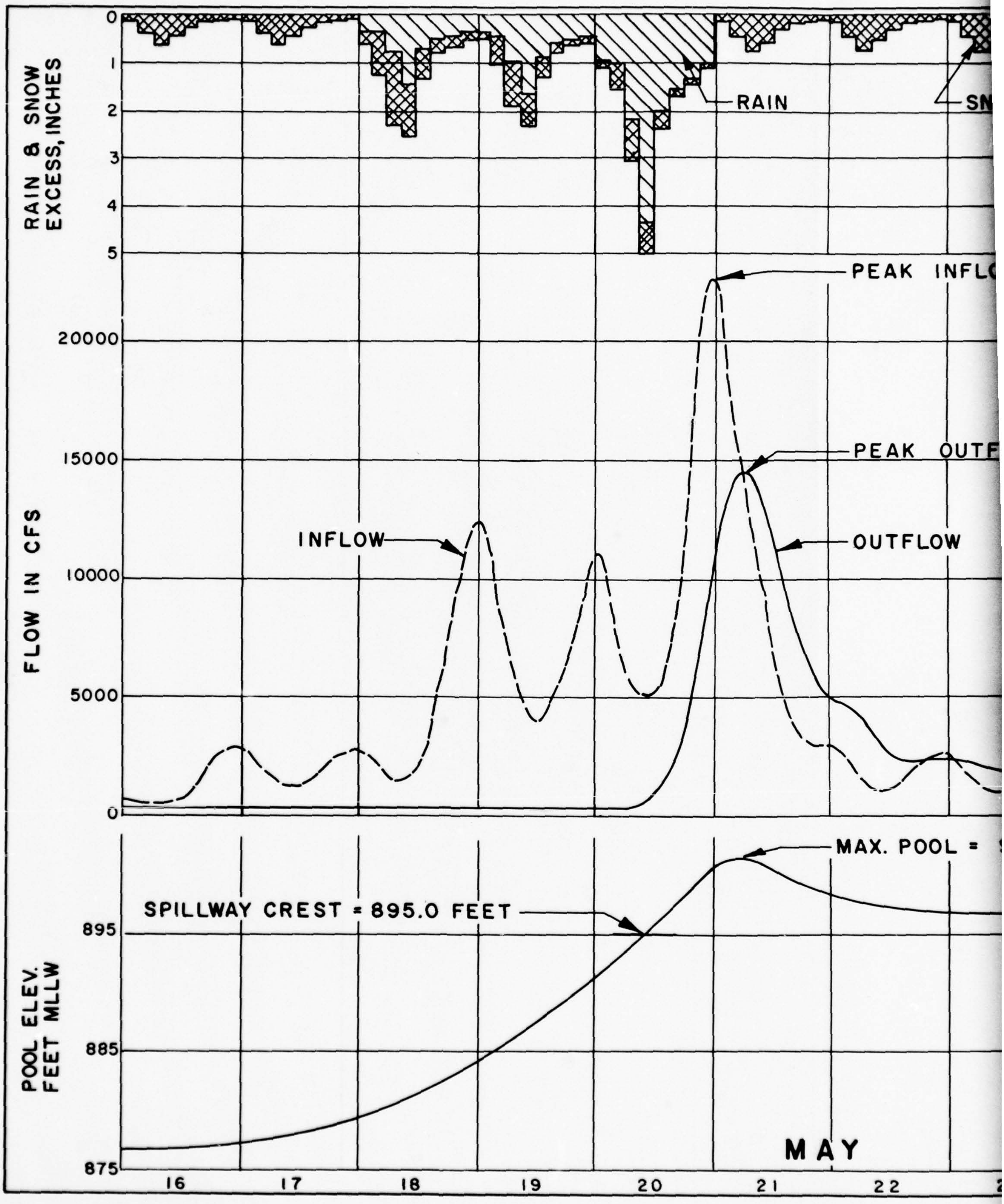
SPILLWAY DESIGN FLOOD

U. S. ARMY ENGINEER DISTRICT, ALASKA

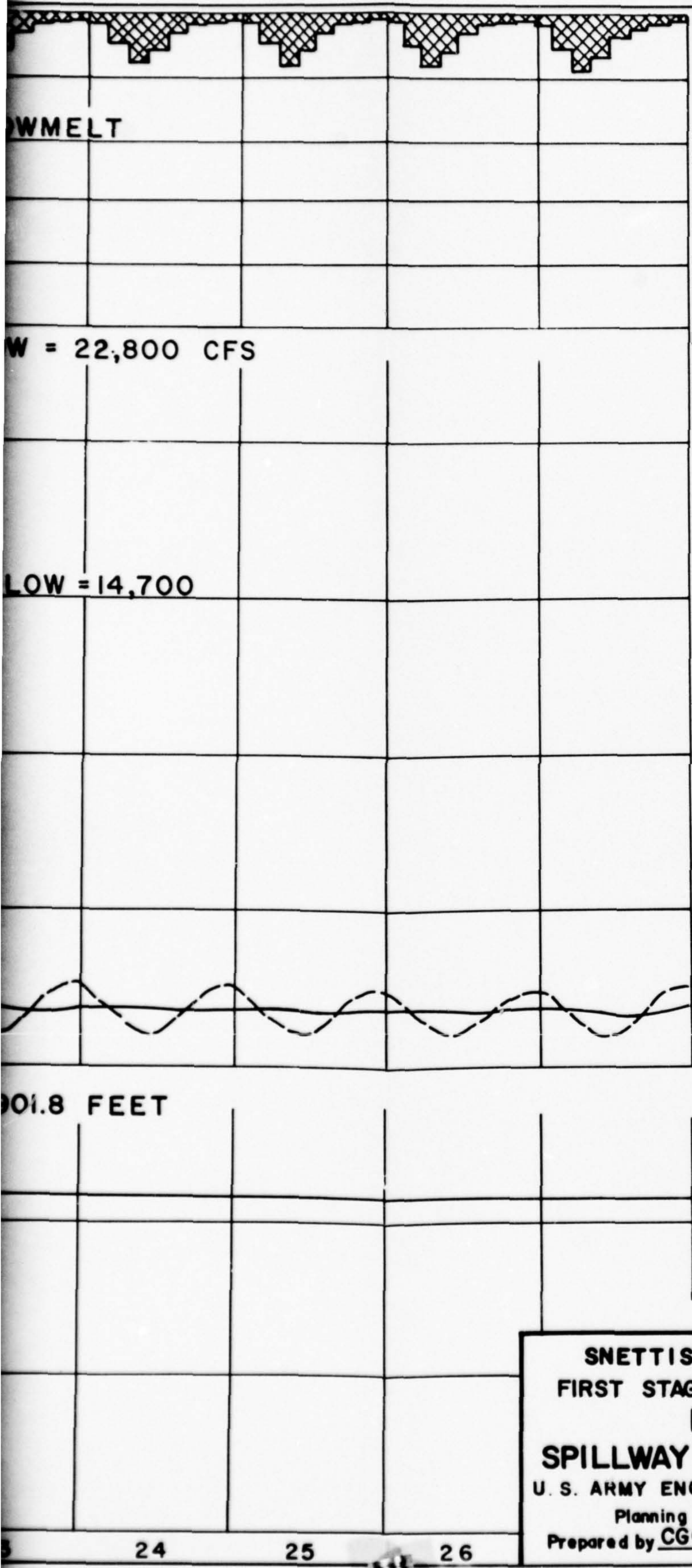
Planning and Reports Branch

Prepared by CGG Date 30 SEP 65

DESIGN MEMORANDUM NO.7 FIGURE 5



RAIN & SNOW EXCESS, INCHES
0
1
2
3
4
5
RAIN
SN
PEAK INFL
20000
15000
PEAK OUTF
INFLOW
OUTFLOW
FLOW IN CFS
10000
5000
0
MAX. POOL =
SPILLWAY CREST = 895.0 FEET
895
POOL ELEV. FEET MLLW
885
875
MAY
16
17
18
19
20
21
22



NOTES

1. INITIAL BASE FLOW IS 360 cfs.

2. INITIAL POOL ELEVATION IS 875 FEET.

3. INITIAL SNOWPACK WATER EQUIVALENT IS 52 INCHES.

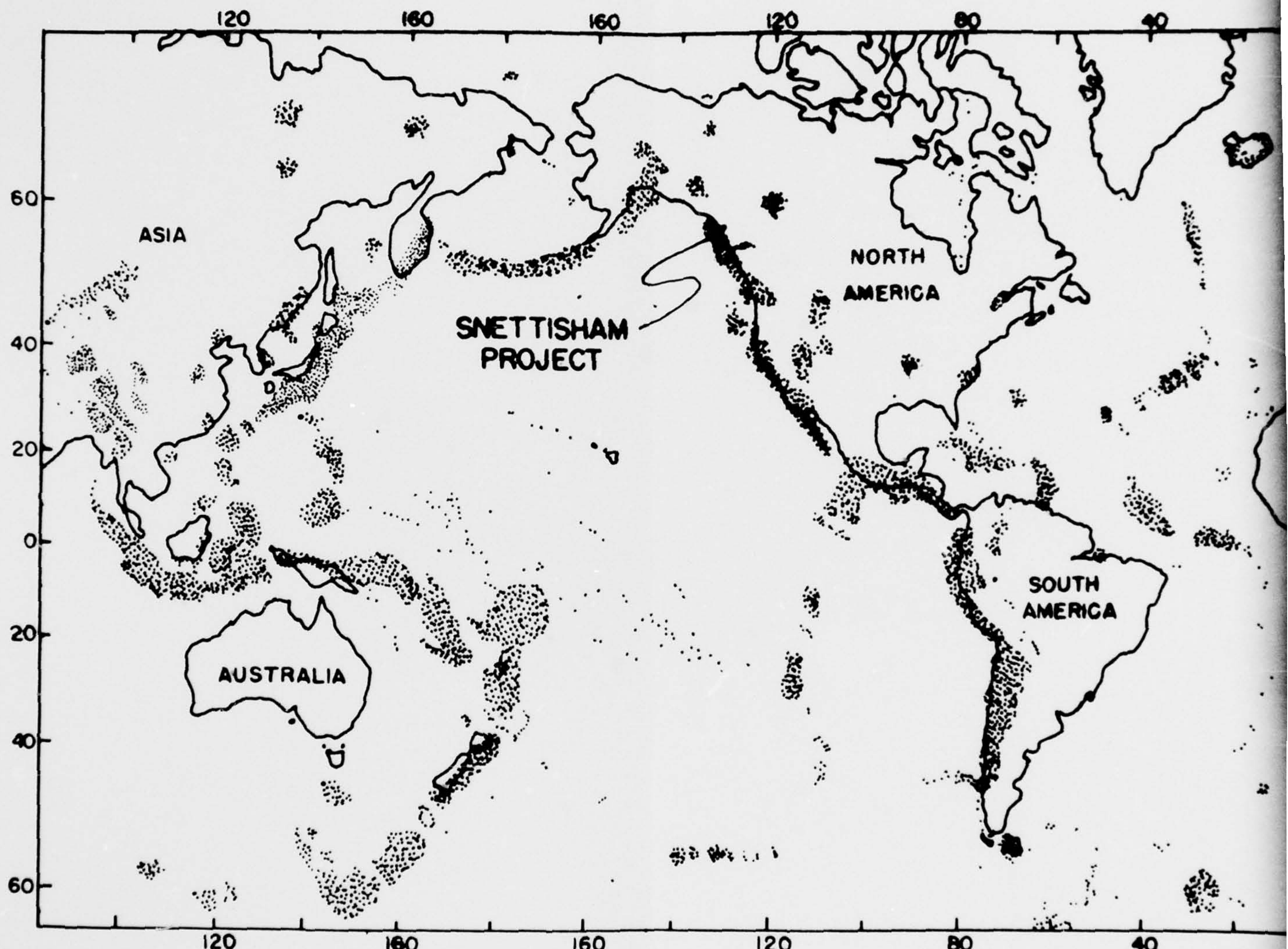
4. THREE DAY PROBABLE MAXIMUM PRECIPITATION IS 24.4 INCHES AND IS GIVEN IN 3 HOUR INCREMENTS ON THE HYETOGRAPH.

5. LOSSES ARE CONSIDERED NEGLIGABLE.

SNETTISHAM PROJECT
FIRST STAGE DEVELOPMENT
MAY
SPILLWAY DESIGN FLOOD
U. S. ARMY ENGINEER DISTRICT, ALASKA
Planning and Reports Branch
Prepared by CGG Date 30 SEP 65

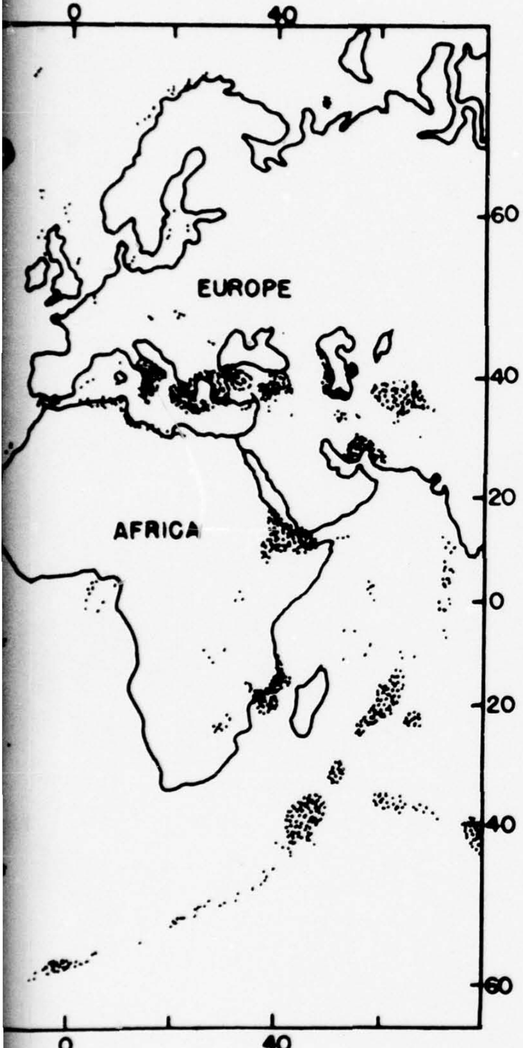
DESIGN MEMORANDUM NO.7 FIGURE 6

COAST AND GEODETIC SURVEY



EARTHQUAKE BELTS OF THE WO

INDIVIDUAL
NECESSAR

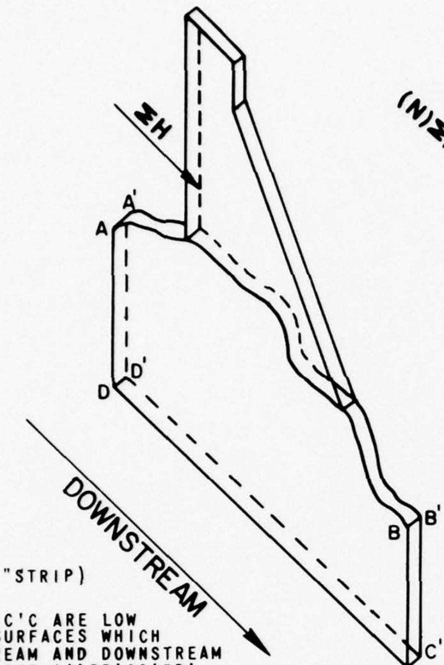


RLD

DOTS ON ABOVE MAP REPRESENT PATTERNS, NOT
RILY SPECIFIC EARTHQUAKE OCCURRENCES

FIGURE 7

2



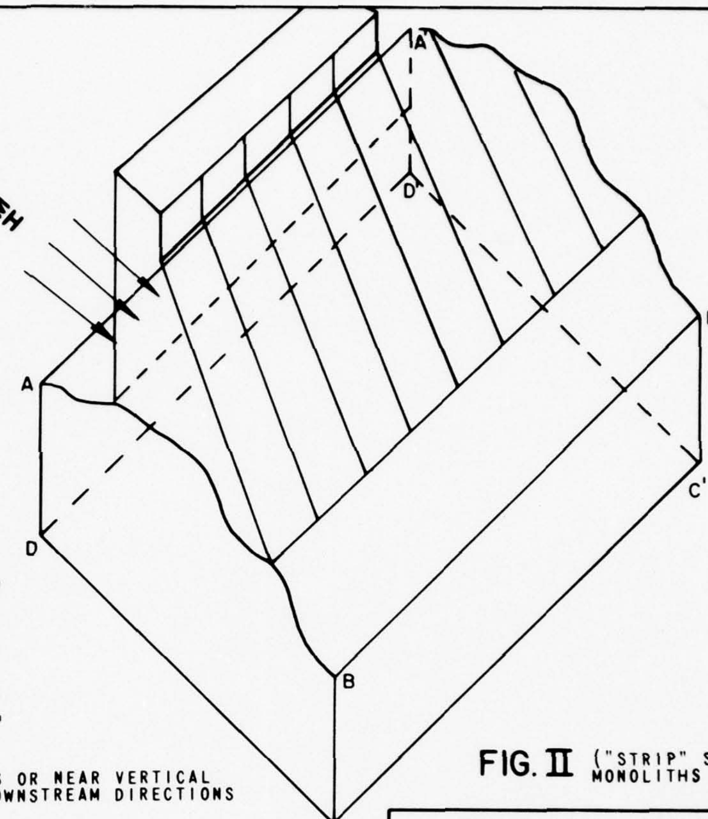


FIG. II ("STRIP" SEVERAL MONOLITHS WIDE)

(1 FOOT WIDE "STRIP)

(1) PLANES DD'C'C ARE LOW ANGLED JOINT SURFACES WHICH DAYLIGHT UPSTREAM AND DOWNSTREAM ON DAM SITE RIDGE AA'BB'CC'DD'

(2) PLANES ABCD ANDA'B'C'D' ARE HIGH ANGLED JOINTS OR NEAR VERTICAL SHEAR PLANES ORIENTED IN APPROXIMATELY UPSTREAM-DOWNSTREAM DIRECTIONS THRU THE ROCK RIDGE

(3) TENDENCY FOR MOVEMENT IN DOWNSTREAM DIRECTION(ΣH)IS RESISTED BY FRICTION AND/OR SHEAR ALONG THE PLANES AS INDICATED BELOW:

$\Sigma H \approx$ DD'CC' + 2ABCD - - - IN FIG I

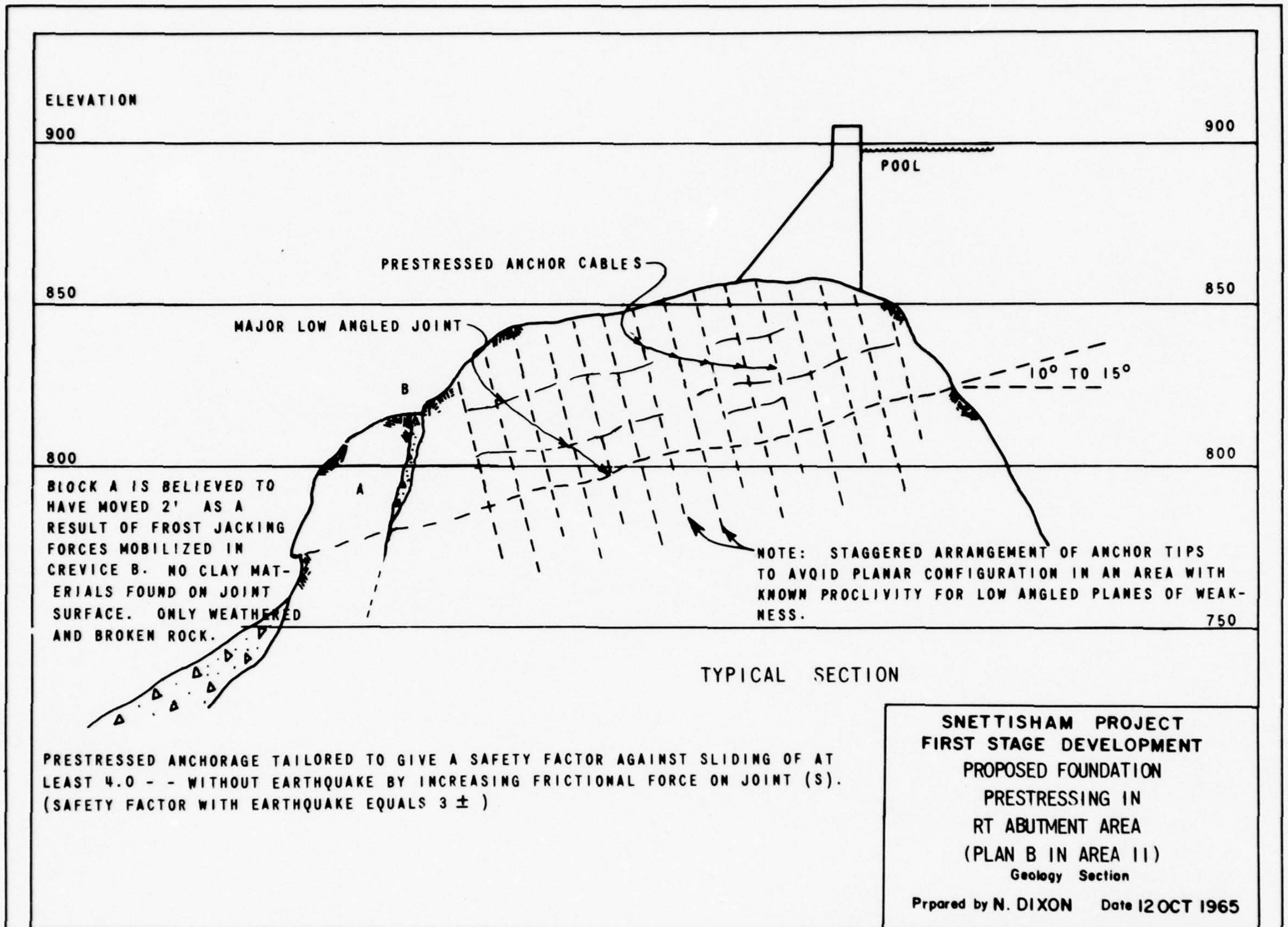
$N\Sigma H \approx N$DD'CC' + 2ABCD - - - IN FIG II

(4) THIS DEMONSTRATES THAT FRICTION AND/OR SHEAR RESISTANCE MUST BE DEVELOPED PRINCIPALLY ALONG THE LOW ANGLE JOINT SURFACES, RATHER THAN VERTICAL PLANES THROUGH THE ROCK.

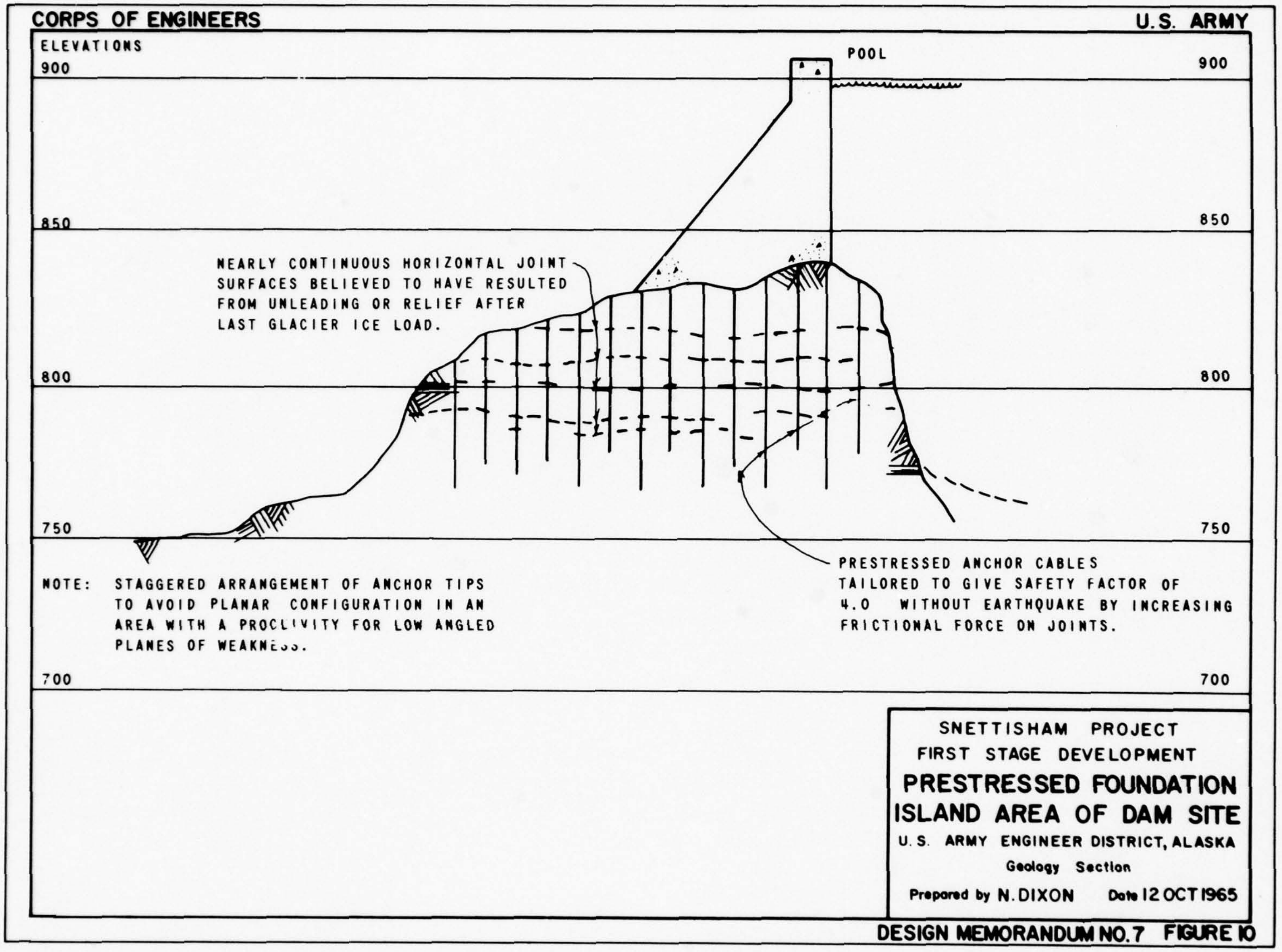
SNETTISHAM PROJECT
FIRST STAGE DEVELOPMENT
IMPORTANCE OF DEVELOPING FULL SLIDING RESISTANCE ALONG LOW ANGLED JOINTS ON A NARROW ROCK RIDGE DAM SITE

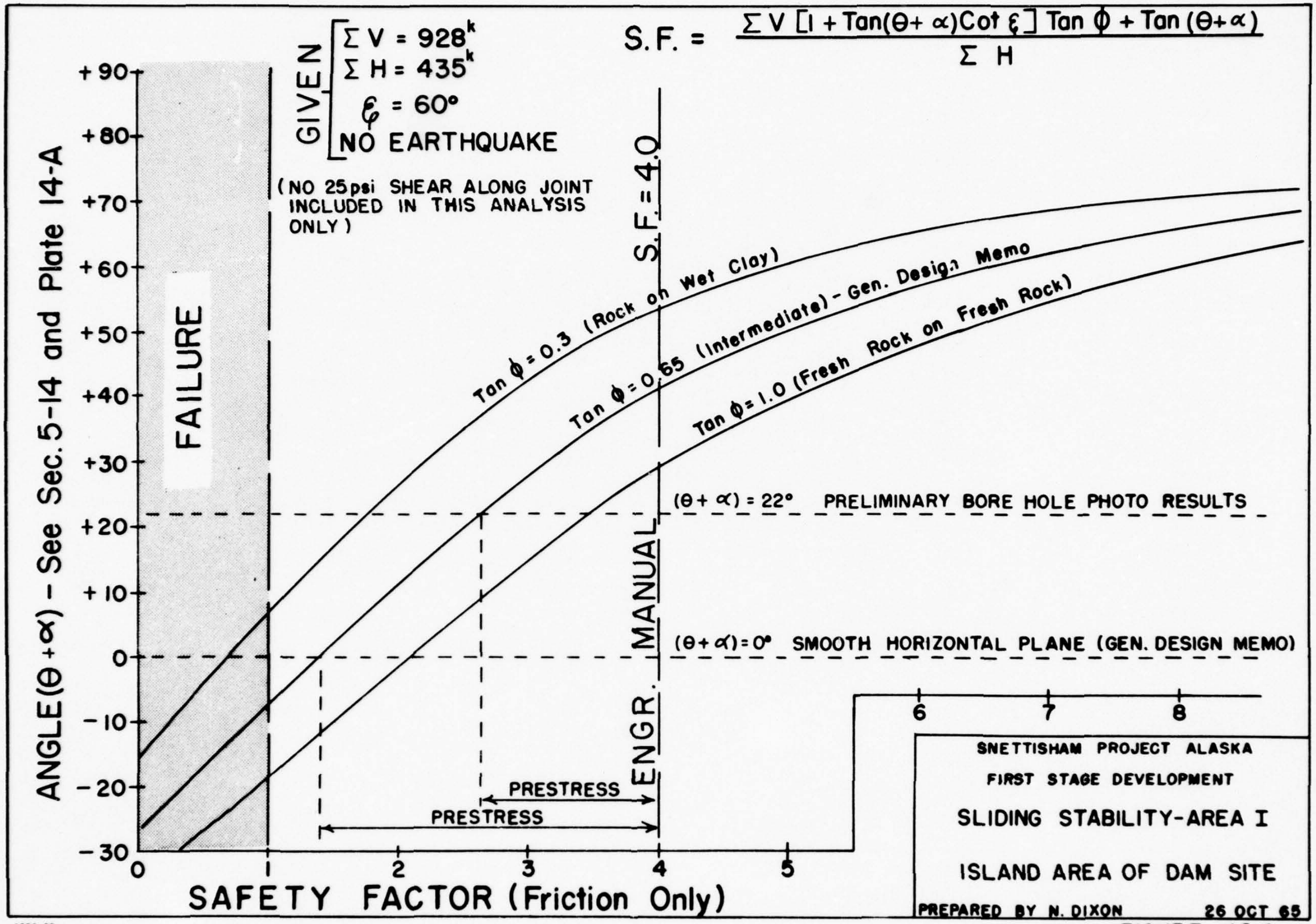
U.S. ARMY ENGINEER DISTRICT, ALASKA
Geology Section
Prepared by N. DIXON Date 12 OCT 1965

DESIGN MEMORANDUM NO.7 FIGURE 8



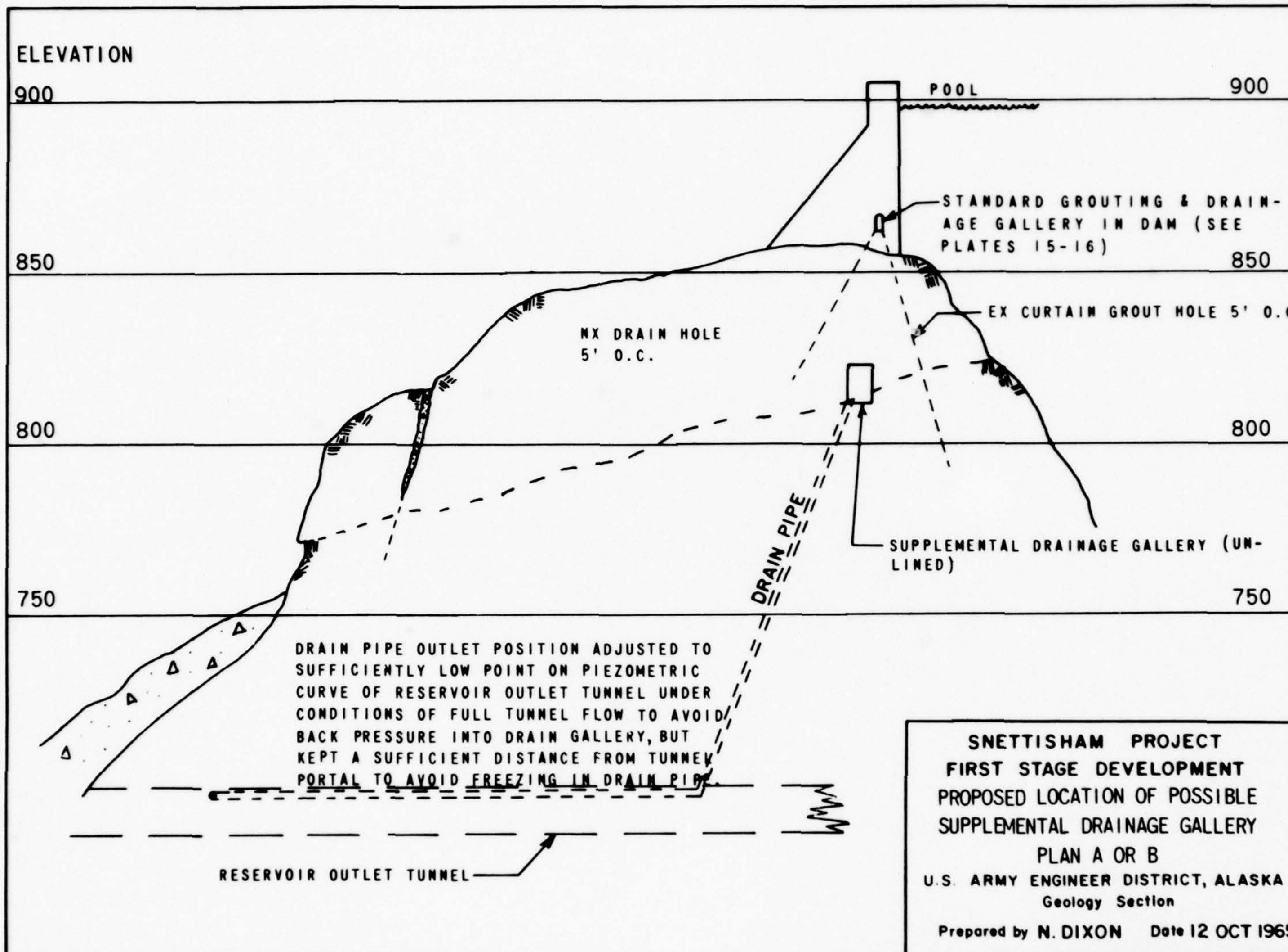
DESIGN MEMORANDUM NO.7 FIGURE 9


CORPS OF ENGINEERS
U.S. ARMY
ELEVATIONS
900
850
800
750
700
POOL
NEARLY CONTINUOUS HORIZONTAL JOINT SURFACES BELIEVED TO HAVE RESULTED FROM UNLEADING OR RELIEF AFTER LAST GLACIER ICE LOAD.
PRESTRESSED ANCHOR CABLES TAILORED TO GIVE SAFETY FACTOR OF 4.0 WITHOUT EARTHQUAKE BY INCREASING FRICTIONAL FORCE ON JOINTS.
NOTE: STAGGERED ARRANGEMENT OF ANCHOR TIPS TO AVOID PLANAR CONFIGURATION IN AN AREA WITH A PROCLIVITY FOR LOW ANGLED PLANES OF WEAKNESS.
SNETTISHAM PROJECT
FIRST STAGE DEVELOPMENT
PRESTRESSED FOUNDATION
ISLAND AREA OF DAM SITE
U.S. ARMY ENGINEER DISTRICT, ALASKA
Geology Section
Prepared by N. DIXON Date 12 OCT 1965
DESIGN MEMORANDUM NO. 7 FIGURE 10



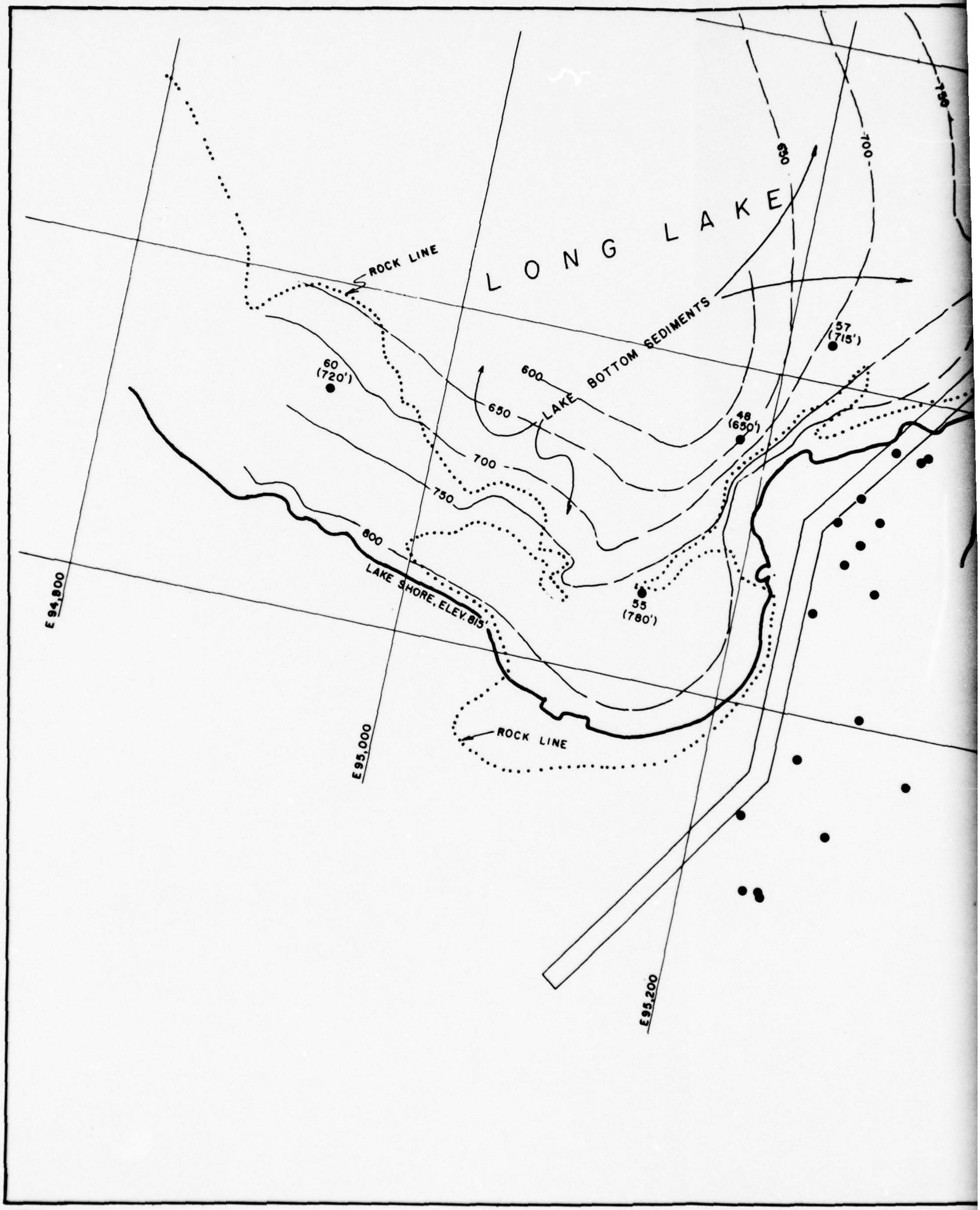
S.F. = $\frac{\Sigma V [1 + \tan(\theta + \alpha)\cot \xi] \tan \phi + \tan(\theta + \alpha)}{\Sigma H}$
GIVEN
$\Sigma V = 928^k$
$\Sigma H = 435^k$
$\xi = 60°$
NO EARTHQUAKE
(NO 25psi SHEAR ALONG JOINT INCLUDED IN THIS ANALYSIS ONLY)
FAILURE
S.F. = 4.0
ENGR. MANUAL
Tan ϕ = 0.3 (Rock on Wet Clay)
Tan ϕ = 0.65 (Intermediate) - Gen. Design Memo
Tan ϕ = 1.0 (Fresh Rock on Fresh Rock)
$(\theta + \alpha) = 22°$ PRELIMINARY BORE HOLE PHOTO RESULTS
$(\theta + \alpha) = 0°$ SMOOTH HORIZONTAL PLANE (GEN. DESIGN MEMO)
PRESTRESS
PRESTRESS
ANGLE $(\theta + \alpha)$ - See Sec. 5-14 and Plate 14-A
+90
+80
+70
+60
+50
+40
+30
+20
+10
0
-10
-20
-30
0
1
2
3
4
5
6
7
8
SAFETY FACTOR (Friction Only)
SNETTISHAM PROJECT ALASKA
FIRST STAGE DEVELOPMENT
SLIDING STABILITY-AREA I
ISLAND AREA OF DAM SITE
PREPARED BY N. DIXON 26 OCT 65
1326-66

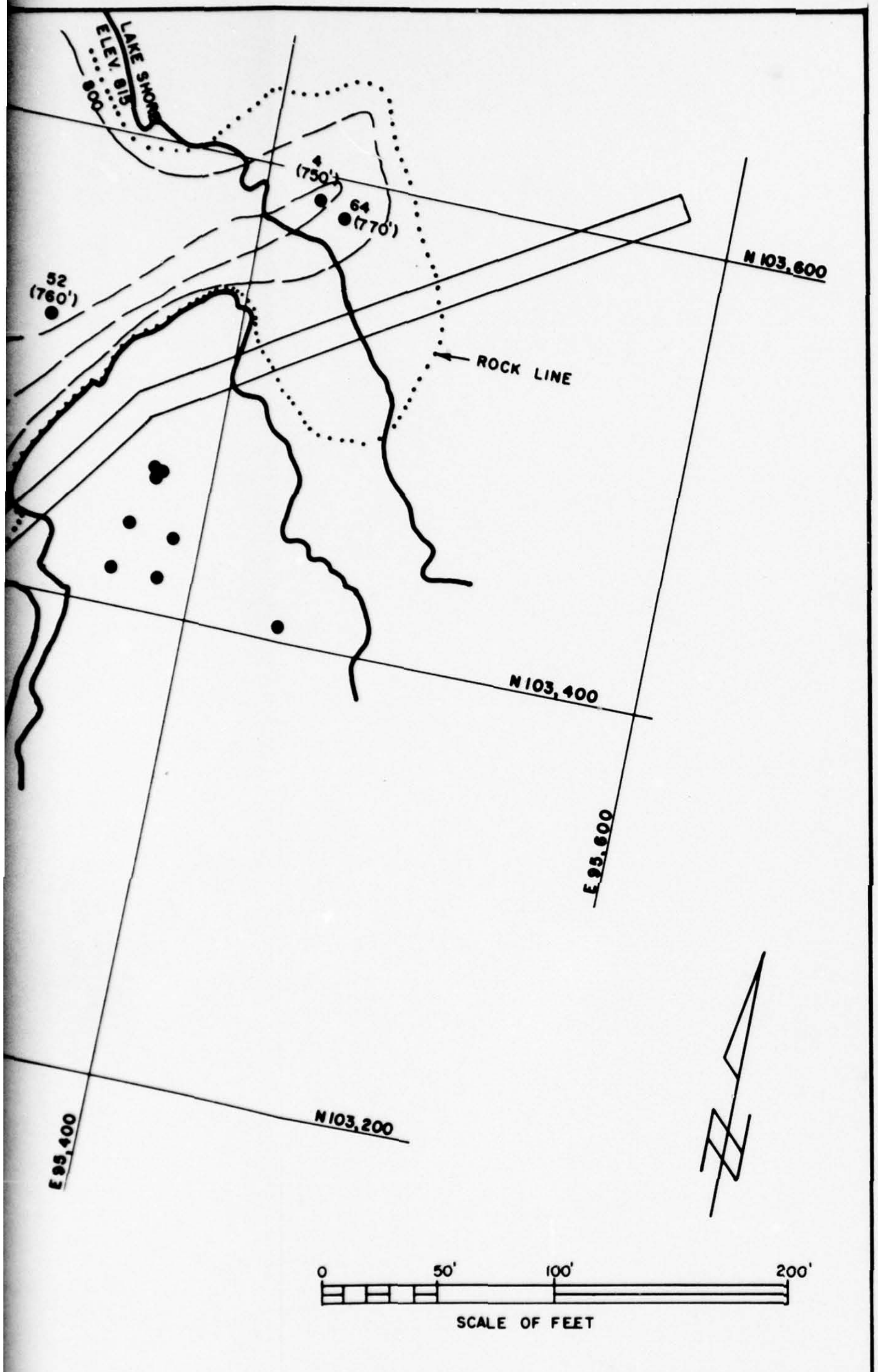
FIGURE 10 A



SNETTISHAM PROJECT
FIRST STAGE DEVELOPMENT
PROPOSED LOCATION OF POSSIBLE SUPPLEMENTAL DRAINAGE GALLERY
PLAN A OR B
U.S. ARMY ENGINEER DISTRICT, ALASKA
Geology Section
Prepared by N. DIXON Date 12 OCT 1965

DESIGN MEMORANDUM NO.7 FIGURE 11


LONG LAKE
ROCK LINE
LAKE BOTTOM SEDIMENTS
600
650
700
750
800
690
700
750
60
(720')
57
(715')
48
(650')
55
(780')
LAKE SHORE, ELEV. 813'
ROCK LINE
E 94,800
E 95,000
E 95,200



SNETTISHAM PROJECT

LONG LAKE DAM SITE

Underwater Bedrock Contours

Bedrock exposed: contours based on lead-line soundings.

Bedrock covered by lake-bottom sediments: contours based on drill-hole data.

55 (780')

Drill hole, with elevation of bedrock

Fig. II-a

A. The following are loading conditions as outlined in EM1110-2-2200 para 3-01 with applicable modifications.

I <u>Construction Condition</u>: Dam completed but no water in the reservoir, wind load acting on the downstream face.

II <u>Normal Operating Conditions</u>: Pool elevation at the top of the spillway crest with ice load acting.

III <u>Induced Surcharge Condition</u>: Not applicable.

IV <u>Flood Discharge Condition</u>: Reservoir at maximum flood pool elevation, no ice pressure-

V <u>Construction Condition with Earthquake</u>: Earthquake acceleration in a downstream direction. No Water in the reservoir and no tail water.

VI <u>Normal Operating Condition with Earthquake</u>: Earthquake acceleration in an upstream direction. Reservoir at the top of the spillway crest with no ice pressure.

B. Use the following design criteria:

a. Unit weight of water = 62.5# / ft^3

b. Unit weight of concrete = 150# / ft^3

c. Ice load 5'-0" thickness @ 5 kip per ft. depth = 25 kip acting 2.5 ft below the spillway crest/per ft width.

d. Wind pressure = 30# / ft^2

e. Earthquake acceleration = 0.1 g. in horizontal direction

f. Period of vibration = 1 sec.

g. Uplift

(1) For dam structures founded on rock a value of zero uplift at the toe varying linearly to 66 2/3% of the full hydrostatic pool pressure at the heel.

(2) Within the concrete a zero value of uplift at the toe varying linearly to 50% of the full hydrostatic pressure at the heel.

(3) Within the joint planes of the foundation a zero value of uplift at the assumed toe position varying linearly to 75% of the upstream pool pressure at the assumed heel.

h. Shear friction factor of safety

(1) The following equation was used where the dam is founded on sound rock or planes through the concrete.

$$S(s-f) = \frac{rsA}{\Sigma H}$$

S(s-f) = Shear friction factor of safety
r = 1.0 (ratio of the average to the maximum shearing stress)
s = 500 psi (allowable shear of the concrete)
A = area of the base
ΣH = sum of the horizontal forces
S(s-f) = 4 minimum

(2) The above equation was modified where low angle joints in the rock foundation occurred.

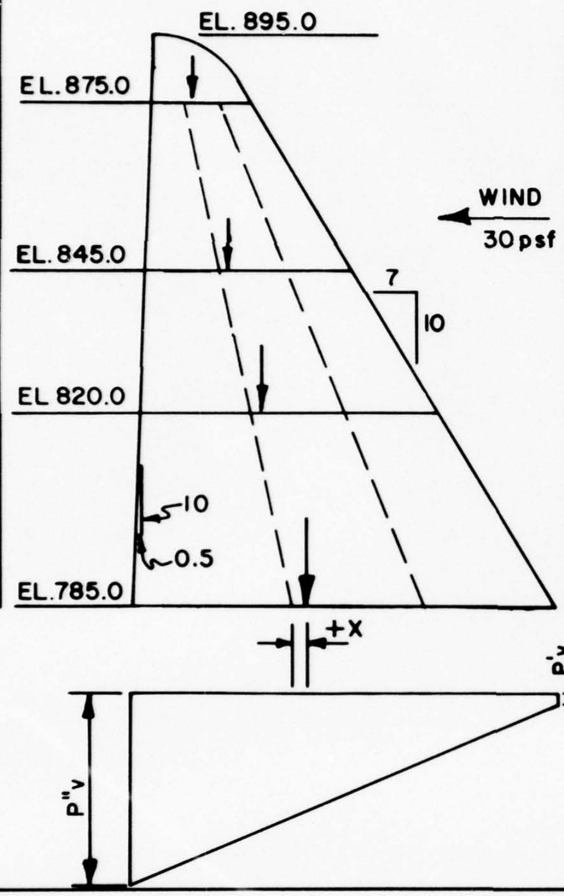
$$Ss\text{-}f = \frac{f\Sigma V + rsA}{\Sigma H}$$

f = 0.65 sliding factor
ΣV sum of the vertical forces
r = 1.0
s = 15 psi near the right abutment (Area II)
s = 25 psi at the central island portion (Area I)
Ss-f = 3 minimum for case VI
Ss-f = 4 minimum case I, II, III, IV, and V

PLAN A & B

STABILITY ANALYSIS — LOADING CONDITION I

	UPSTREAM				DOWNSTREAM			GENERAL					
Elevation	P"i	P"v	P"n		P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
EL. 875.0	3.35	3.34	0.0		1.20	0.81	0.0	0.0	54.9	0.600	0.011	3200	+1.3
EL. 845.0	8.31	8.30	0.0		1.53	1.01	0.0	0.0	228.0	1.5	0.007	2350	+1.8
EL 820.0	11.96	11.95	0.0		1.80	1.21	0.0	0.0	436.0	2.3	0.005	2160	+2.0
EL. 785.0	17.20	17.10	0.0		1.93	1.30	0.0	0.0	870.0	3.30	0.004	2050	+2.3



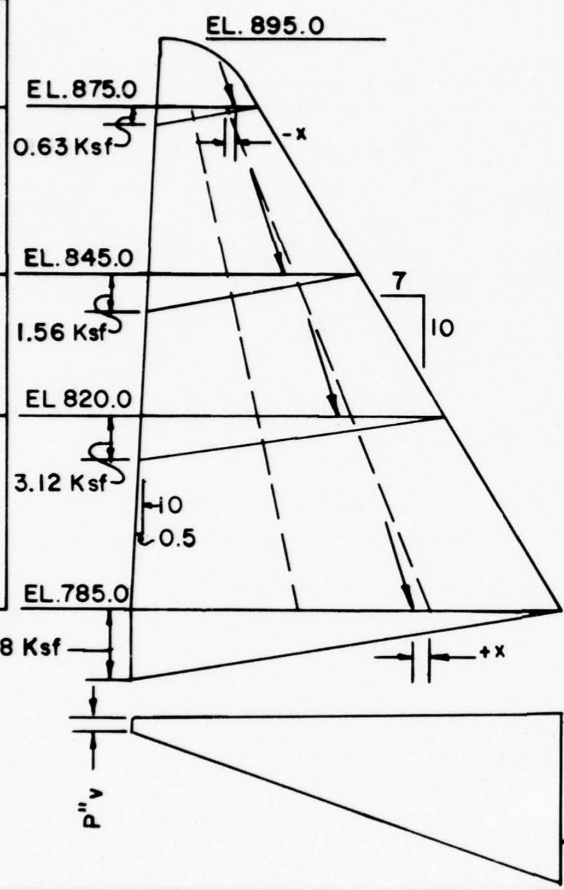
LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (-); (ft)
Pi - Maximum principal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

STABILITY ANALYSIS LOADING CONDITION II

UPSTREAM				DOWNSTREAM			GENERAL					
P''i	P''v	P''n	Elevation	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
-1.56	-1.55	1.25	EL. 875.0	7.51	5.04	0.0	8.3	46.6	37.5	0.804	51.0	-3.9
0.44	0.43	3.12	EL. 845.0	10.95	7.35	0.0	38.2	189.8	103.1	0.545	34.1	+0.9
0.72	0.71	4.68	EL 820.0	13.85	9.30	0.0	106.1	340.0	201.0	0.594	24.2	+1.7
2.05	2.04	6.88	EL. 785.0	18.30	12.30	0.0	216.0	672.7	403.0	0.600	16.8	+4.5



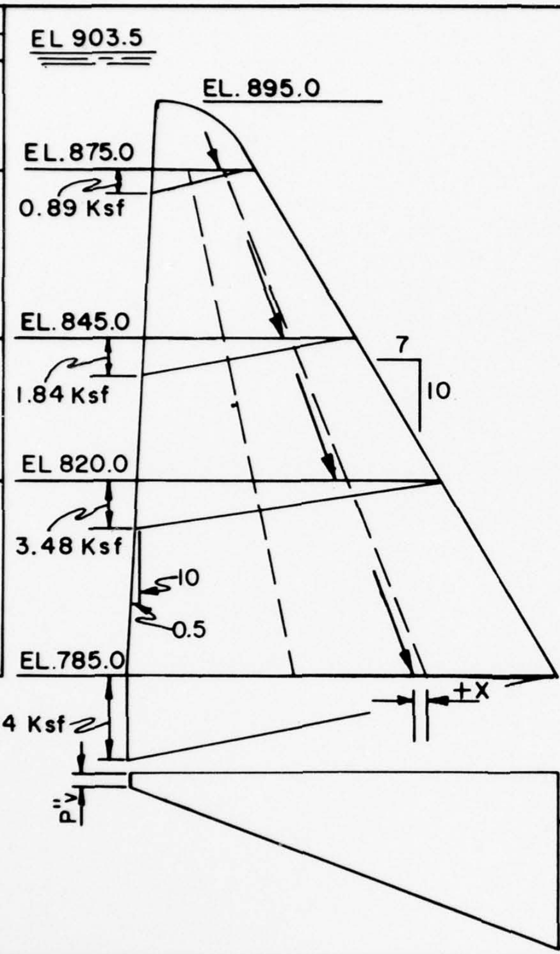
LEGEND

- U – Uplift unit pressure (ksf)
- ΣU – Total uplift on given plane (kips)
- ΣH – Total of all horizontal forces above the given elevations (kips)
- ΣV – Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f – Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. – Shear friction factor
- X – Distance inside middle third (+); outside (–); (ft)
- Pi – Maximum principal stress at face of dam (ksf)
- Pv – Vertical component of stress at face of dam (ksf)
- Pn – Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

STABILITY ANALYSIS LOADING CONDITION IV

UPSTREAM				DOWNSTREAM			GENERAL					
P''i	P''v	P''n	Elevation	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
0.57	0.56	1.78	EL. 875.0	4.03	2.71	0.0	11.8	43.1	25.4	0.590	75.1	+1.5
1.09	1.08	3.68	EL. 845.0	9.50	6.38	0.0	45.3	182.7	108.0	0.594	32.6	+2.4
0.52	0.51	5.22	EL 820.0	13.65	9.15	0.0	118.5	327.5	218.0	0.667	22.4	+1.3
1.20	1.19	7.41	EL. 785.0	19.10	12.80	0.0	232.0	659.4	439.0	0.667	15.4	+2.7



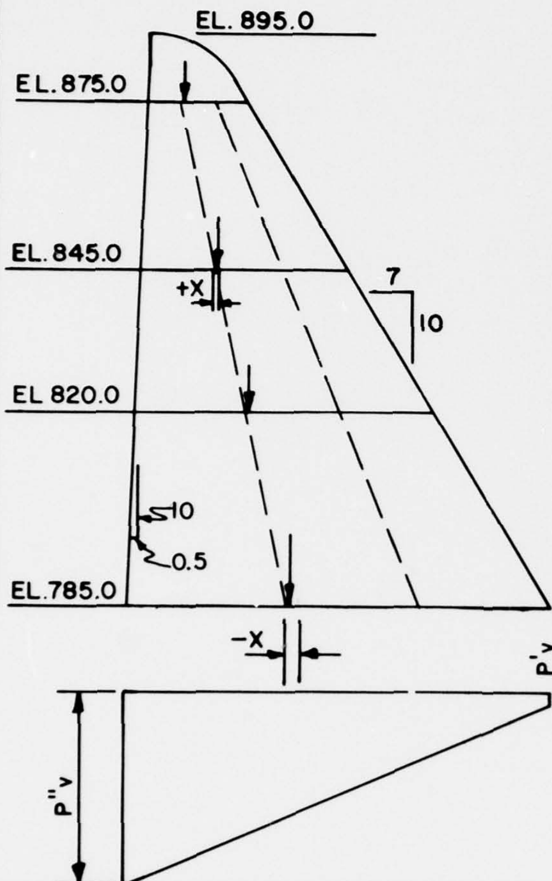
LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (-); (ft)
Pi - Maximum principal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

STABILITY ANALYSIS — LOADING CONDITION V

	UPSTREAM			DOWNSTREAM			GENERAL					
Elevation	P''i	P''v	P''n	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
EL. 875.0	3.84	3.83	0.0	0.39	0.26	0.0	0.0	54.9	5.5	0.100	346.0	+0.6
EL. 845.0	9.30	9.29	0.0	0.03	0.02	0.0	0.0	228.0	22.8	0.100	155.0	+0.02
EL 820.0	13.53	13.52	0.0	0.0	0.0	0.0	0.0	446.0	44.6	0.100	109.0	−0.8
EL. 785.0	19.6	19.50	0.0	0.0	0.0	0.0	0.0	870.0	87.0	0.100	77.8	− 1.5



LEGEND

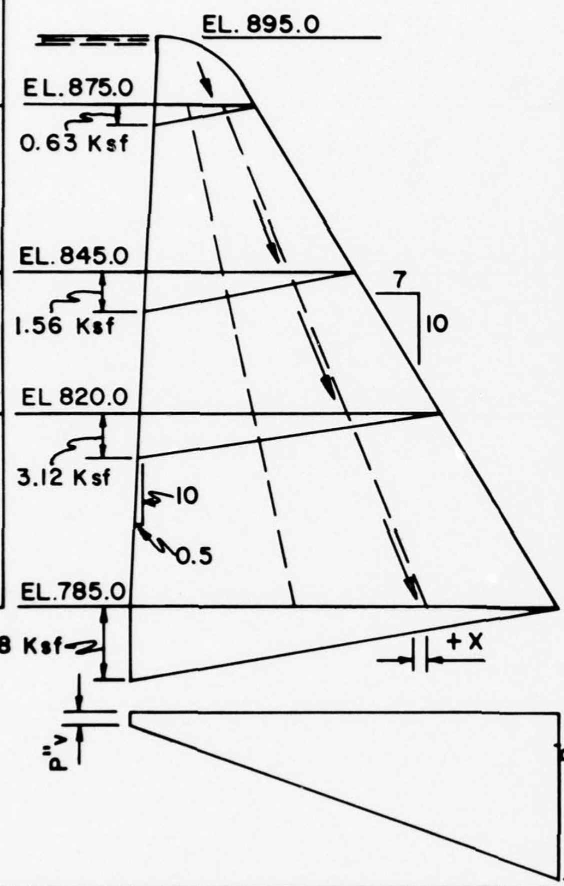
U – Uplift unit pressure (ksf)
ΣU – Total uplift on given plane (kips)
ΣH – Total of all horizontal forces above the given elevations (kips)
ΣV – Total of vertical forces acting on a plane, including the effect of uplift (kips)
f – Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. – Shear friction factor
X – Distance inside middle third (+); outside (−); (ft)
Pi – Maximum principal stress at face of dam (ksf)
Pv – Vertical component of stress at face of dam (ksf)
Pn – Normal component of pressure due to external load at face of dam (ksf)

DESIGN MEMORANDUM NO. 7 FIGURE 13-4

PLAN A & B

STABILITY ANALYSIS · LOADING CONDITION VI

UPSTREAM				DOWNSTREAM			GENERAL					
P''i	P''v	P''n	Elevation	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
1.57	1.56	1.33	EL. 875.0	2.96	1.99	0.0	8.3	46.6	21.2	0.455	90.0	+3.9
1.65	1.64	3.50	EL. 845.0	9.10	6.10	0.0	38.2	189.8	113.6	0.600	31.0	+3.5
0.49	0.48	5.14	EL 820.0	14.25	9.56	0.0	106.0	340.0	243.6	0.716	200.0	+ 1.2
0.21	0.20	7.44	EL. 785.0	21.20	14.25	0.0	216.0	672.7	506.4	0.753	13.4	+ 0.3



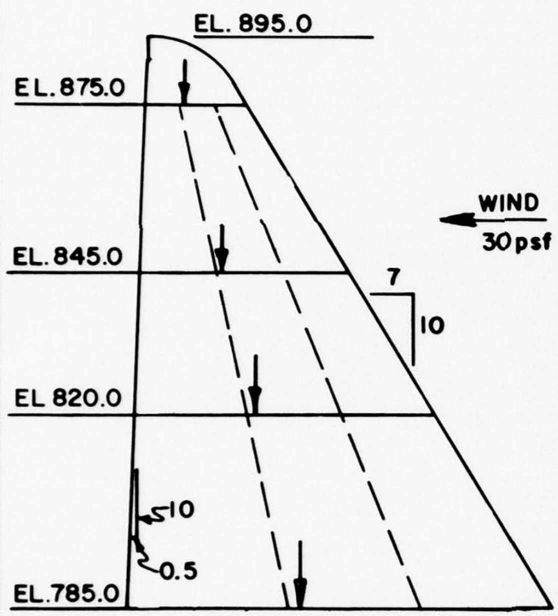
LEGEND

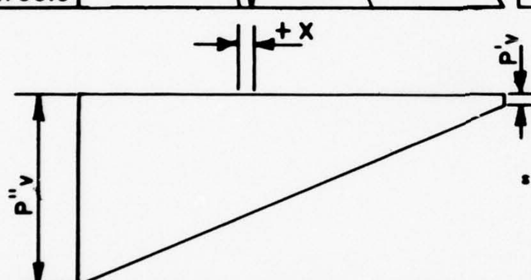
U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (−); (ft)
Pi - Maximum principal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN C

STABILITY ANALYSIS LOADING CONDITION I

	UPSTREAM			DOWNSTREAM			GENERAL					
Elevation	P''i	P''v	P''n	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
EL. 875.0	3.35	3.34	0.0	1.20	0.81	0.0	0.0	54.9	0.600	0.011	3200	+ 1.3
EL. 845.0	8.31	8.30	0.0	1.53	1.01	0.0	0.0	228.0	1.5	0.007	2350	+ 1.8
EL 820.0	11.96	11.95	0.0	1.80	1.21	0.0	0.0	436.0	2.3	0.005	2160	+ 2.0
EL. 785.0	17.20	17.10	0.0	1.83	1.30	0.0	0.0	870.0	3.3	0.004	2050	+ 2.3





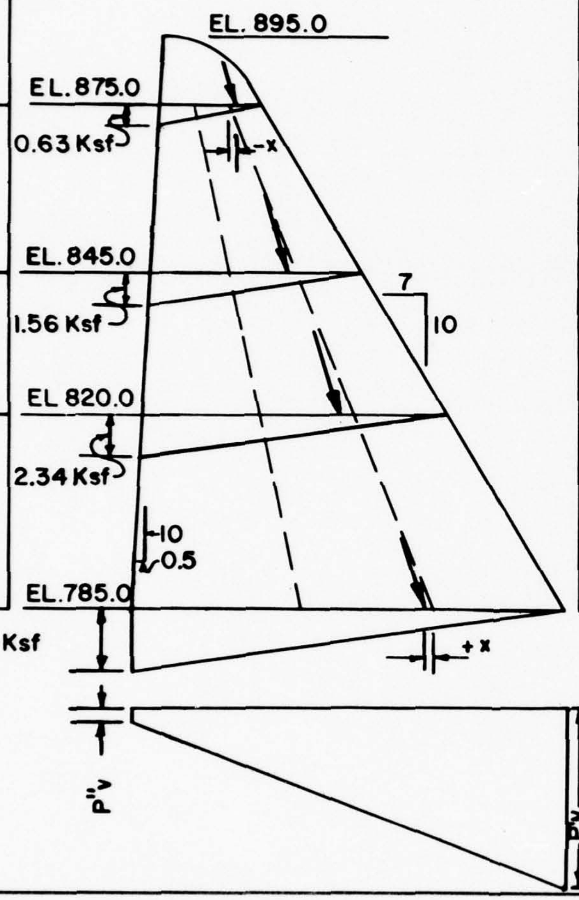
LEGEND

- U – Uplift unit pressure (ksf)
- ΣU – Total uplift on given plane (kips)
- ΣH – Total of all horizontal forces above the given elevations (kips)
- ΣV – Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f – Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. – Shear friction factor
- X – Distance inside middle third (+); outside (–); (ft)
- Pi – Maximum principal stress at face of dam (ksf)
- Pv – Vertical component of stress at face of dam (ksf)
- Pn – Normal component of pressure due to external load at face of dam (ksf)

PLAN C

STABILITY ANALYSIS LOADING CONDITION II

UPSTREAM			DOWNSTREAM			GENERAL					
P''i	P''v	P''n	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
-1.56	-1.55	1.25	7.51	5.84	0.0	8.3	46.6	37.5	0.804	51.0	-3.9
0.44	0.43	3.12	10.95	7.35	0.0	38.2	189.8	103.1	0.545	34.1	+0.9
1.48	1.47	4.68	13.77	9.30	0.01	79.4	366.6	201.0	0.548	24.2	+3.2
2.05	2.04	6.88	18.30	12.30	0.0	216.0	672.7	403.0	0.600	16.8	+4.5



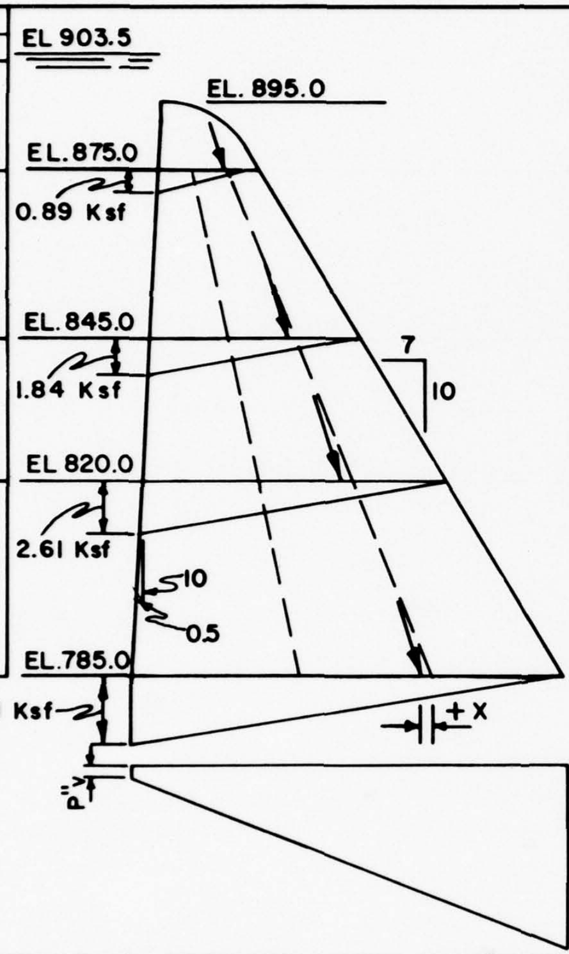
LEGEND

- U - Uplift unit pressure (ksf)
- ΣU - Total uplift on given plane (kips)
- ΣH - Total of all horizontal forces above the given elevations (kips)
- ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. - Shear friction factor
- X - Distance inside middle third (+); outside (-); (ft)
- Pi - Maximum principal stress at face of dam (ksf)
- Pv - Vertical component of stress at face of dam (ksf)
- Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN C

STABILITY ANALYSIS — LOADING CONDITION IV

UPSTREAM		
P''i	P''v	P''n
0.57	0.56	1.78
1.09	1.08	3.68
1.36	1.35	5.22
1.20	1.19	7.41



DOWNSTREAM			GENERAL					
P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
4.03	2.71	0.0	11.8	43.1	25.4	0.590	75.1	+1.5
9.50	6.38	0.0	45.3	182.7	108.0	0.594	32.6	+2.4
13.70	9.20	0.0	88.4	357.6	218.0	0.610	22.4	+30
19.10	12.80	0.0	232.0	655.4	439.0	0.667	15.4	+2.7

LEGEND

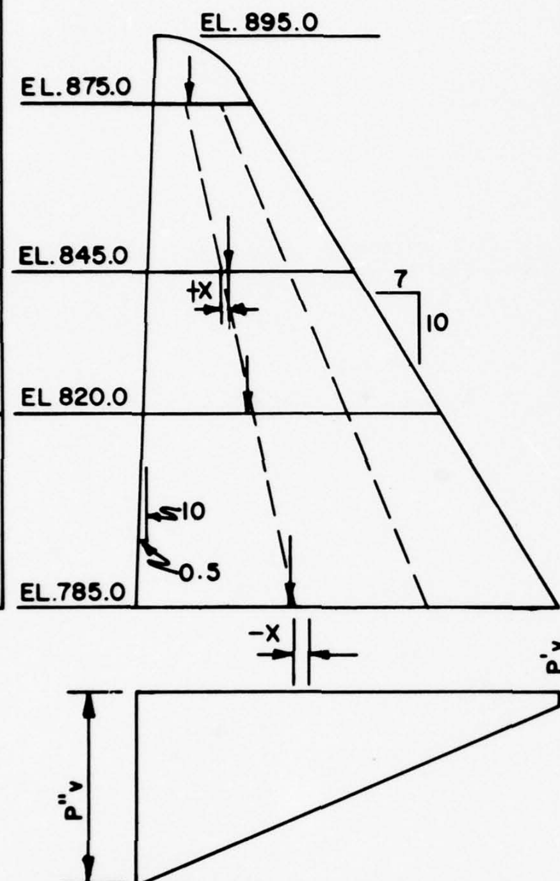
- U - Uplift unit pressure (ksf)
- ΣU - Total uplift on given plane (kips)
- ΣH - Total of all horizontal forces above the given elevations (kips)
- ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. - Shear friction factor
- X - Distance inside middle third (+); outside (-); (ft)
- Pi - Maximum principal stress at face of dam (ksf)
- Pv - Vertical component of stress at face of dam (ksf)
- Pn - Normal component of pressure due to external load at face of dam (ksf)

DESIGN MEMORANDUM NO. 7 FIGURE 14-3

PLAN C

STABILITY ANALYSIS LOADING CONDITION V

UPSTREAM			Elevation	DOWNSTREAM			GENERAL					
P''i	P''v	P''n		P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
3.84	3.83	0.0	EL. 875.0	0.39	0.26	0.0	0.0	54.9	5.5	0.100	346.0	+ 0.6
9.30	9.29	0.0	EL. 845.0	0.03	0.02	0.0	0.0	228.0	22.8	0.100	155.0	+ 0.02
13.53	13.52	0.0	EL 820.0	0.0	0.0	0.0	0.0	446.0	44.6	0.100	109.0	− 0.8
19.6	19.50	0.0	EL. 785.0	0.0	0.0	0.0	0.0	870.0	87.0	0.100	77.8	− 1.5



LEGEND

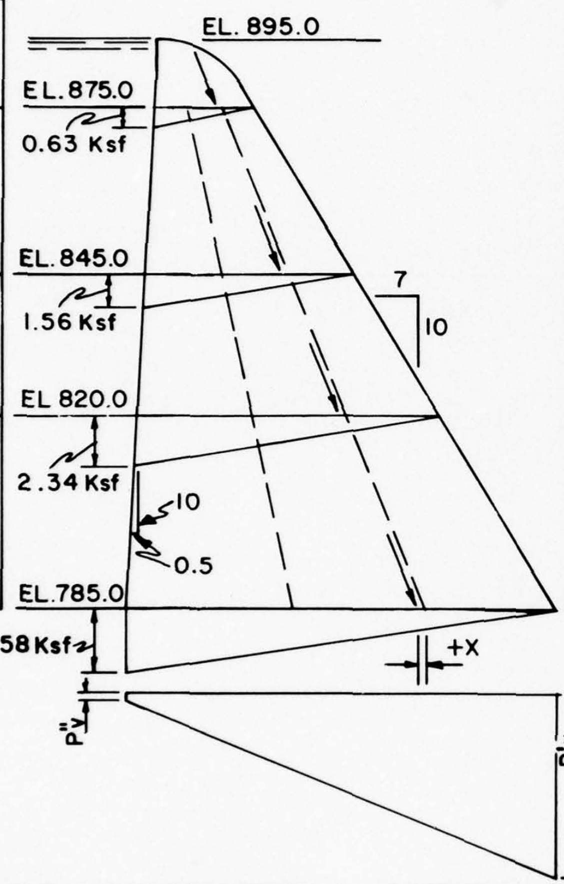
U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (−); (ft)
Pi - Maximum principal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

DESIGN MEMORANDUM NO. 7 FIGURE 14-4

PLAN C

STABILITY ANALYSIS LOADING CONDITION VI

UPSTREAM			DOWNSTREAM			GENERAL					
P"i	P"v	P"n	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
1.57	1.56	1.33	2.96	1.99	0.0	8.3	46.6	21.2	0.455	90.0	+ 3.9
1.65	1.64	3.50	9.10	6.10	0.0	38.2	189.8	113.6	0.600	31.0	+ 3.5
1.26	1.25	5.14	14.27	9.57	0.0	79.4	366.6	243.8	0.664	20.0	+ 2.7
0.21	0.20	7.44	21.20	14.25	0.0	216.0	672.7	506.4	0.753	13.4	+ 0.3



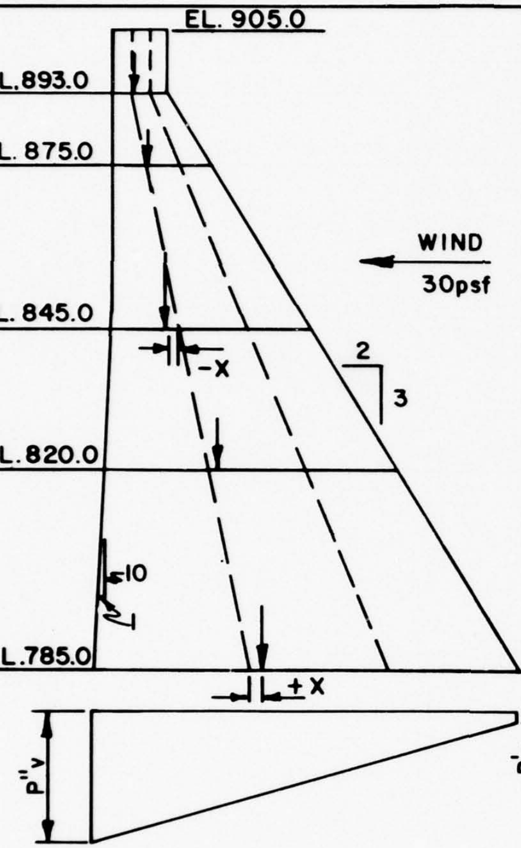
LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (-); (ft)
Pi - Maximum principal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN A&B

STABILITY ANALYSIS LOADING CONDITION I

Upstream P"i	Upstream P"v	Upstream P"n	Elevation	Downstream P'i	Downstream P'v	Downstream P'n	General ΣU	General ΣV	General ΣH	General f	General S.F.	General X
1.89	1.89	0.0	EL. 893.0	1.71	1.71	0.0	0.0	21.6	0.36	0.017	2,400	+1.9
5.80	5.80	0.0	EL. 875.0	0.11	0.07	0.0	0.0	70.3	0.9	0.013	1,920	+0.1
10.35	10.35	0.0	EL. 845.0	0	0	0.0	0.0	223.0	1.80	0.008	1,730	-0.3
12.95	12.80	0.0	EL. 820.0	0.96	0.67	0.0	0.0	425.0	2.6	0.006	1,785	+1.0
16.90	16.70	0.0	EL. 785.0	2.39	1.66	0.0	0.0	826.0	3.6	0.004	1800	+2.7



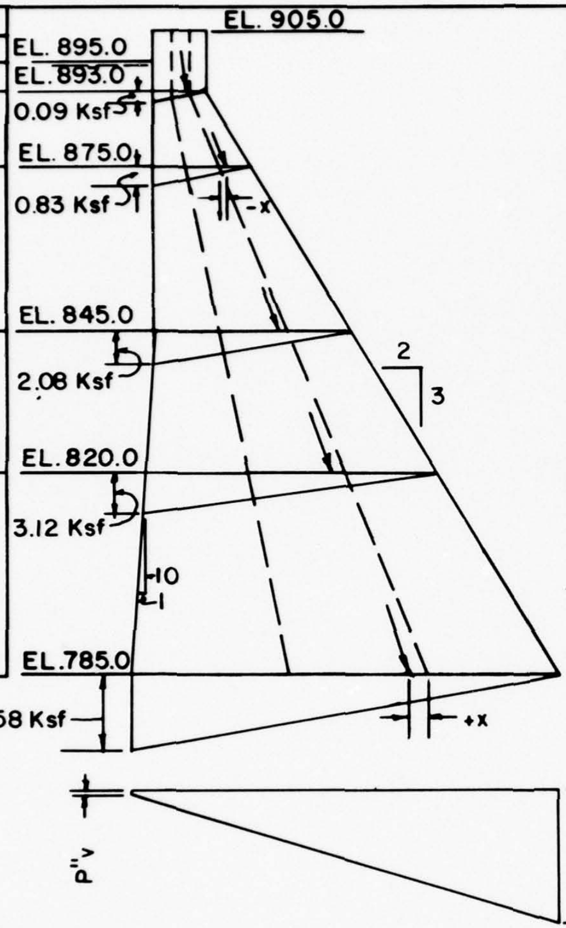
LEGEND

- U - Uplift unit pressure (ksf)
- ΣU - Total uplift on given plane (kips)
- ΣH - Total of all horizontal forces above the given elevations (kips)
- ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. - Shear friction factor
- X - Distance inside middle third (+); outside (-); (ft)
- Pi - Maximum pricipal stress at face of dam (ksf)
- Pv - Vertical component of stress at face of dam (ksf)
- Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

STABILITY ANALYSIS LOADING CONDITION II

UPSTREAM				DOWNSTREAM			GENERAL					
P''i	P''v	P''n	Elevation	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
1.34	1.34	0.13	EL. 893.0	3.21	2.22	0.0	0.5	21.1	10.1	0.478	85.3	+1.5
0.0	0.0	1.25	EL. 875.0	8.32	5.75	0.0	10.0	60.3	32.5	0.622	40.3	-1.0
0.04	0.04	3.12	EL. 845.0	11.07	7.66	0.0	45.8	177.2	103.1	0.582	30.7	+0.7
0.84	0.83	4.68	EL. 820.0	14.20	9.81	0.0	98.5	336.3	201.0	0.598	22.6	+1.6
0.09	0.09	6.88	EL. 785.0	19.50	13.50	0.0	207.0	649.0	403.0	0.623	16.0	+1.9



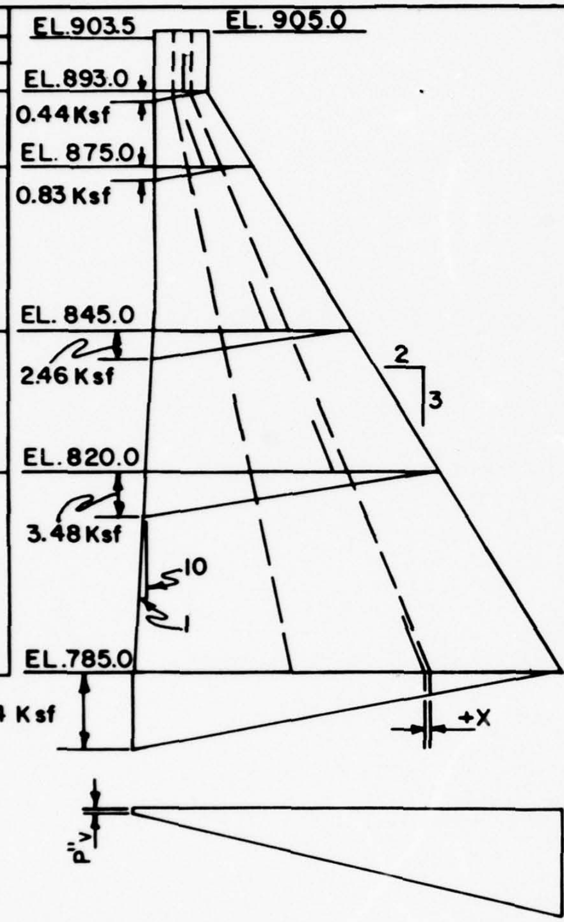
LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (-); (ft)
Pi - Maximum pricipal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

STABILITY ANALYSIS — LOADING CONDITION IV

UPSTREAM				DOWNSTREAM			GENERAL					
P"i	P"v	P"n	Elevation	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
0.87	0.87	0.66	EL. 893.0	3.33	2.30	0.0	2.6	19.0	3.5	0.183	249.0	+1.1
1.93	1.93	1.25	EL. 875.0	3.98	2.75	0.0	14.2	56.1	25.4	0.453	68.0	+3.3
1.22	1.22	3.68	EL. 845.0	9.36	6.48	0.0	54.0	169.0	107.0	0.633	29.6	+2.3
0.07	0.07	5.22	EL. 820.0	13.93	9.65	0.0	110.0	326.0	218.0	0.669	20.8	+1.4
0.14	0.14	7.41	EL. 785.0	20.20	14.00	0.0	222.0	637.2	439.0	0.690	14.8	+0.1



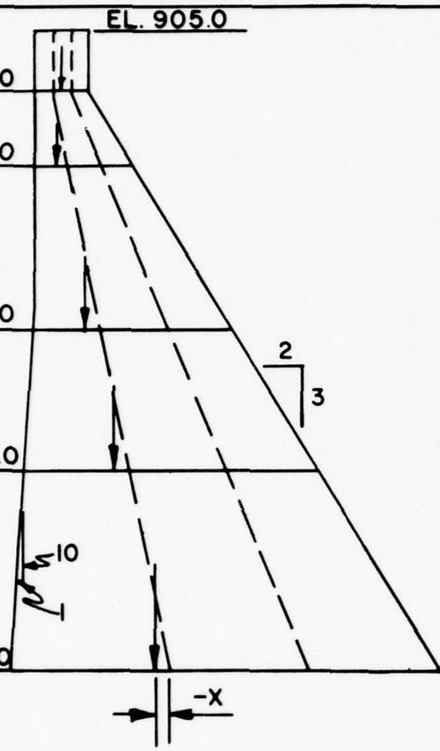
LEGEND

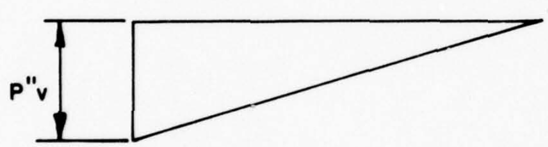
- U – Uplift unit pressure (ksf)
- ΣU – Total uplift on given plane (kips)
- ΣH – Total of all horizontal forces above the given elevations (kips)
- ΣV – Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f – Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. – Shear friction factor
- X – Distance inside middle third (+); outside (−); (ft)
- Pi – Maximum pricipal stress at face of dam (ksf)
- Pv – Vertical component of stress at face of dam (ksf)
- Pn – Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

STABILITY ANALYSIS LOADING CONDITION V

UPSTREAM				DOWNSTREAM			GENERAL					
P"i	P"v	P"n		P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
2.34	2.34	0.0	EL. 893.0	-1.26	1.26	0.0	0.0	21.6	2.2	0.100	400.0	+ 1.3
6.75	6.75	0.0	EL. 875.0	0.0	0.0	0.0	0.0	70.3	7.0	0.100	215.0	−1.0
12.1	12.1	0.0	EL. 845.0	0.0	0.0	0.0	0.0	223.0	22.3	0.100	119.0	−2.4
14.83	14.68	0.0	EL. 820.0	0.0	0.0	0.0	0.0	425.0	42.5	0.100	98.0	− 1.8
19.45	19.23	0.0	EL. 785.0	0.0	0.0	0.0	0.0	826.0	82.6	0.100	75.0	−1.3



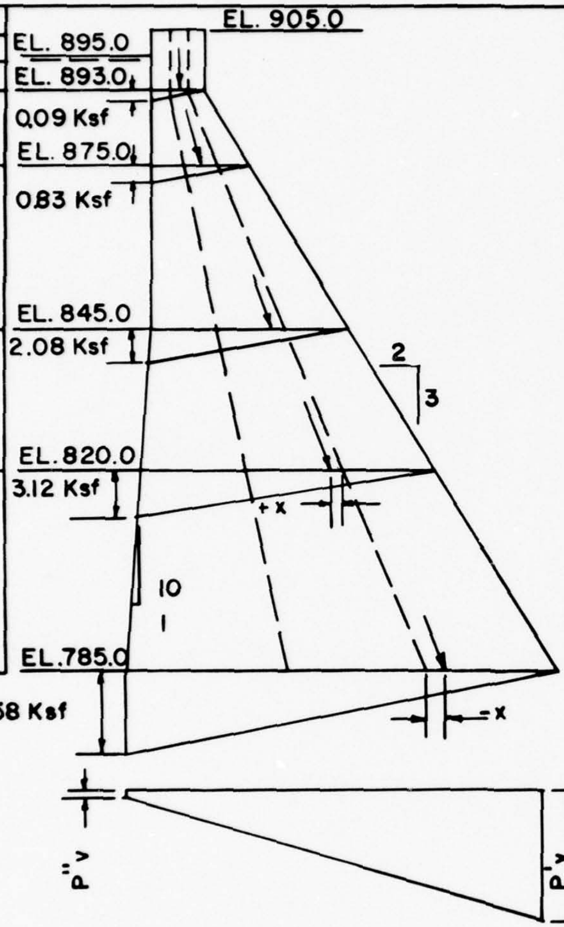


LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (−); (ft)
Pi - Maximum pricipal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

STABILITY ANALYSIS — LOADING CONDITION VI

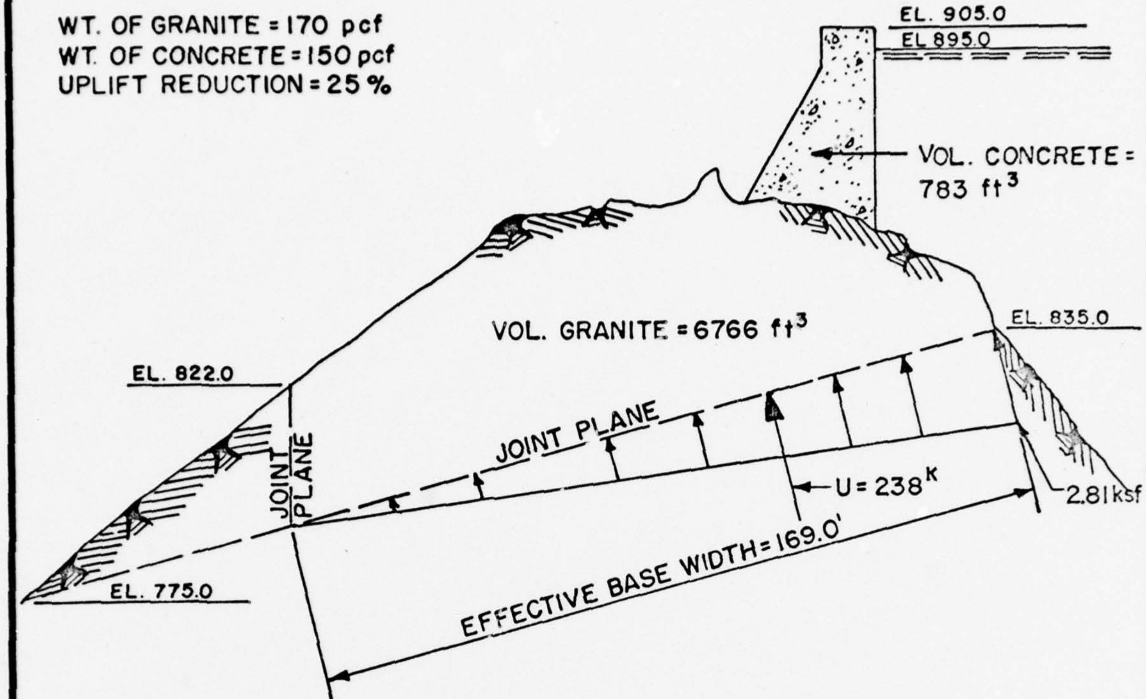


Elevation	Upstream P"i	Upstream P"v	Upstream P"n	Downstream P'i	Downstream P'v	Downstream P'n	General ΣU	General ΣV	General ΣH	General f	General S.F.	General X
EL. 893.0	1.15	1.15	0.21	3.46	2.39	0.0	0.5	21.1	2.4	0.113	361.0	+1.3
EL. 875.0	2.71	2.71	1.49	3.37	2.33	0.0	10.0	60.3	22.7	0.376	76.1	+4.3
EL. 845.0	1.72	1.72	3.50	9.16	6.34	0.0	45.8	177.2	113.0	0.507	28.0	+3.1
EL. 820.0	0.49	0.48	5.14	14.7	10.2	0.0	98.5	336.3	241.7	0.720	18.8	+0.9
EL. 785.0	0.0	0.0	7.44	22.22	15.40	0.0	207.0	649.0	502.0	0.775	12.1	-2.0

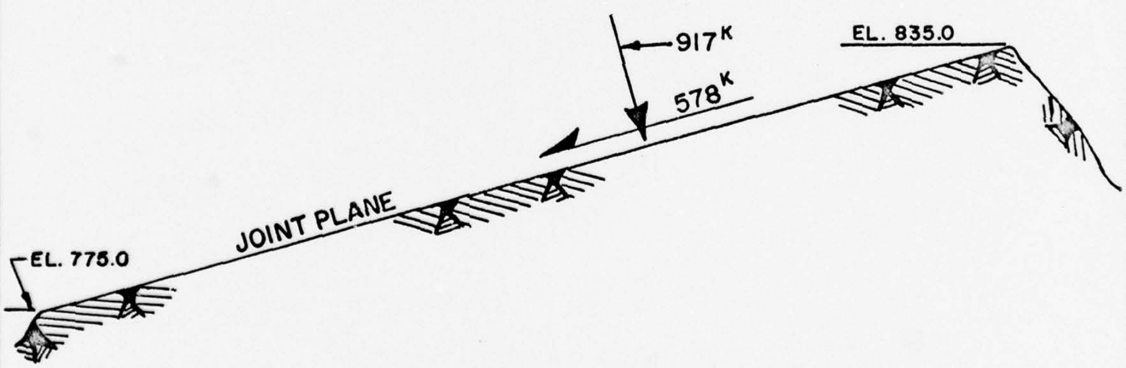
LEGEND

- U - Uplift unit pressure (ksf)
- ΣU - Total uplift on given plane (kips)
- ΣH - Total of all horizontal forces above the given elevations (kips)
- ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. - Shear friction factor
- X - Distance inside middle third (+); outside (-); (ft)
- Pi - Maximum pricipal stress at face of dam (ksf)
- Pv - Vertical component of stress at face of dam (ksf)
- Pn - Normal component of pressure due to external load at face of dam (ksf)

WT. OF GRANITE = 170 pcf
WT. OF CONCRETE = 150 pcf
UPLIFT REDUCTION = 25 %



STATION 6 + 50
MAX. HORIZONTAL FORCES ⟶ CONDITION IV



RESULTANT LOADING DIAGRAM

SHEAR FRICTION FACTOR OF SAFETY

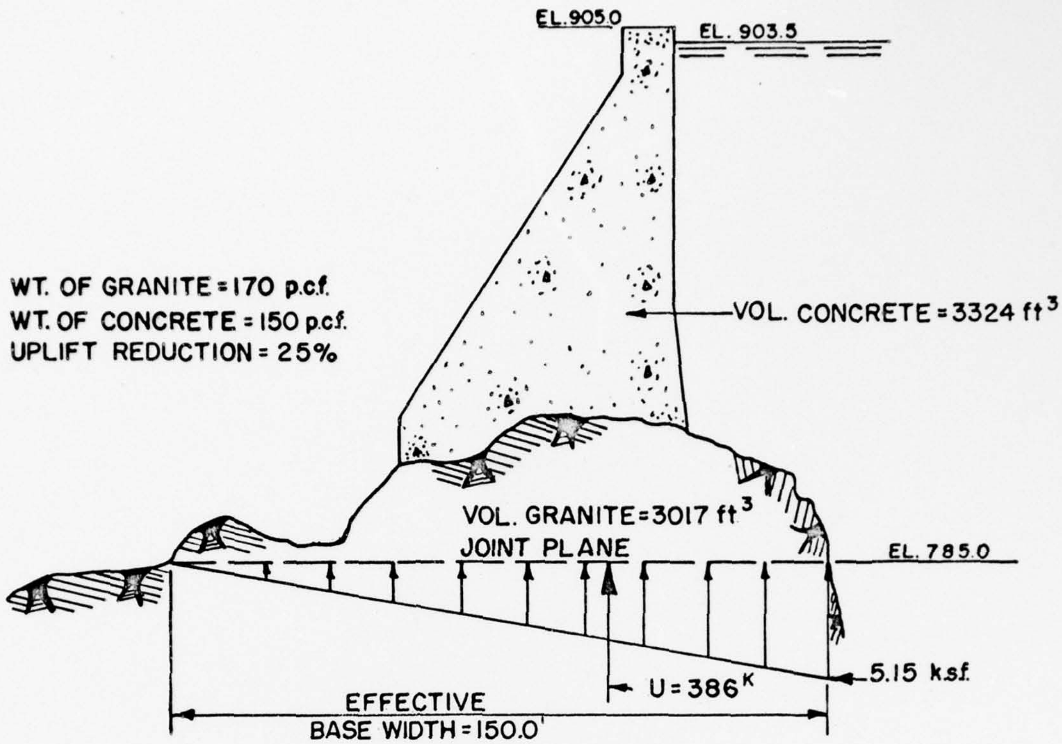
$$= \frac{[(\Sigma V - U)(0.65)] + (b \times q)}{\Sigma H}$$

$$= \frac{[(917)(0.65) + (169 \times 2.16)}{568}$$

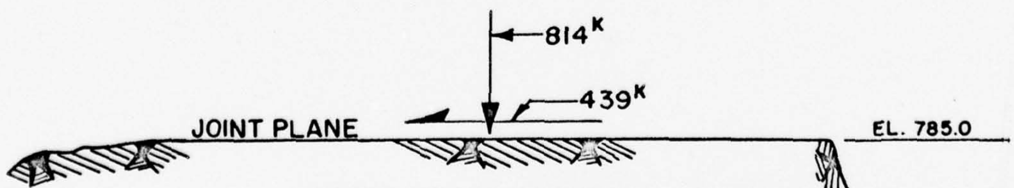
$$= 1.69$$

WHERE $q = 15$ psi
$b = 169'$

PLAN "A"
FOUNDATION STABILITY ANALYSIS
AREA II



STATION 4+00
MAX. HORIZONTAL FORCES ⟶ CONDITION VI



RESULTANT LOADING DIAGRAM

SHEAR FRICTION FACTOR OF SAFETY $= \dfrac{[(\Sigma V - U)(0.65)] + (b \times q)}{\Sigma H}$

$= \dfrac{[(814)(0.65)] + (150)(3.6)}{439}$

$= \underline{\underline{2.43}}$

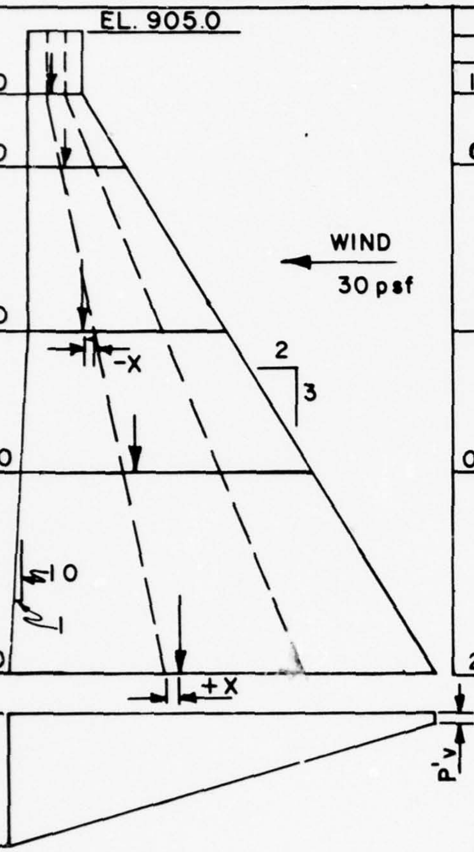
WHERE $q = 25$ psi
$b = 150.0'$

PLAN "A"
FOUNDATION STABILITY ANALYSIS
AREA I

PLAN C

STABILITY ANALYSIS — LOADING CONDITION I

UPSTREAM				DOWNSTREAM			GENERAL					
P"i	P"v	P"n	Elevation	P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
1.89	1.89	0.0	EL. 893.0	1.71	1.71	0.0	0.0	21.6	0.36	0.017	2,400	+1.9
5.80	5.80	0.0	EL. 875.0	0.11	0.07	0.0	0.0	70.3	0.9	0.013	1,920	+0.1
10.35	10.35	0.0	EL. 845.0	0	0	0.0	0.0	223.0	1.80	0.008	1,730	-0.3
12.95	12.80	0.0	EL. 820.0	0.96	0.67	0.0	0.0	425.0	2.6	0.006	1,785	+1.0
16.90	16.70	0.0	EL. 785.0	2.39	1.66	0.0	0.0	826.0	3.6	0.004	1800	+27



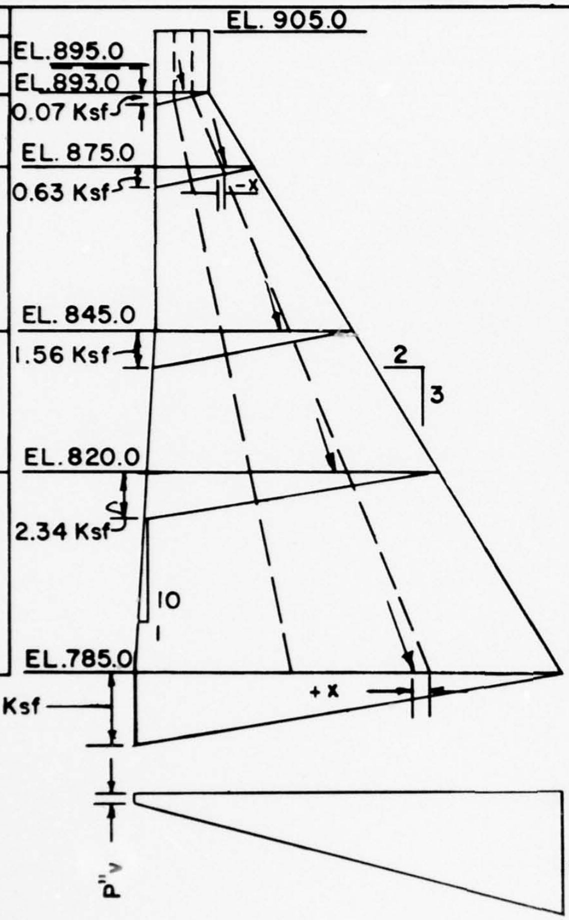
LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (-); (ft)
Pi - Maximum pricipal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN C

STABILITY ANALYSIS LOADING CONDITION II

UPSTREAM				DOWNSTREAM			GENERAL					
P''i	P''v	P''n		P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
1.34	1.34	0.13	EL. 893.0	2.20	2.20	0.0	0.40	21.2	10.1	0.478	85.3	+1.5
0.0	0.0	1.25	EL. 875.0	8.31	5.75	0.0	7.5	62.8	37.5	0.598	42.0	-0.7
0.90	0.90	3.12	EL. 845.0	11.10 -	7.69	0.0	34.3	188.7	103.1	0.546	30.7	+1.6
1.62	1.60	4.68	EL. 820.0	14.20	9.85	0.0	73.7	361.1	201.0	0.557	22.6	+3.0
0.09	0.09	6.88	EL. 785.0	19.50	13.50	0.0	207.0	649.0	403.0	0.623	16.0	+1.9



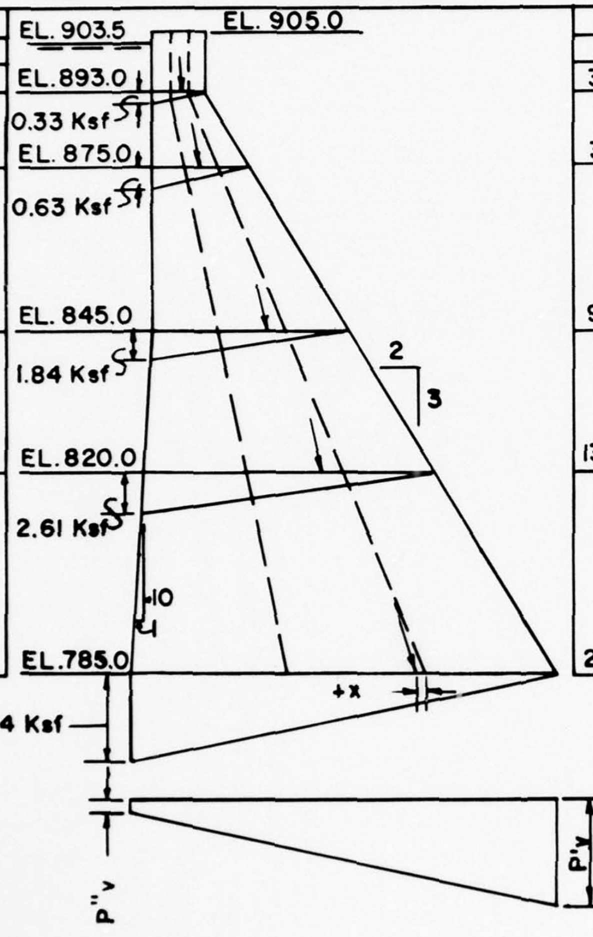
LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (-); (ft)
Pi - Maximum pricipal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN C

STABILITY ANALYSIS — LOADING CONDITION IV

Elevation	Upstream P"i	Upstream P"v	Upstream P"n	Downstream P'i	Downstream P'v	Downstream P'n	General ΣU	General ΣV	General ΣH	General f	General S.F.	General X
EL. 893.0	0.98	0.98	0.66	3.32	2.29	0.0	2.0	19.6	3.5	0.177	249.0	+1.2
EL. 875.0	2.23	2.23	1.25	3.96	2.74	0.0	10.7	59.6	25.4	0.426	68.0	+3.6
EL. 845.0	1.83	1.83	3.68	9.37	6.49	0.0	40.5	182.5	107.0	0.586	29.6	+3.3
EL. 820.0	1.64	1.62	5.22	13.81	9.57	0.0	82.5	353.5	218.0	0.616	20.8	+3.1
EL. 785.0	0.14	0.14	7.41	20.20	14.00	0.0	222.0	637.2	439.0	0.690	14.9	+0.1



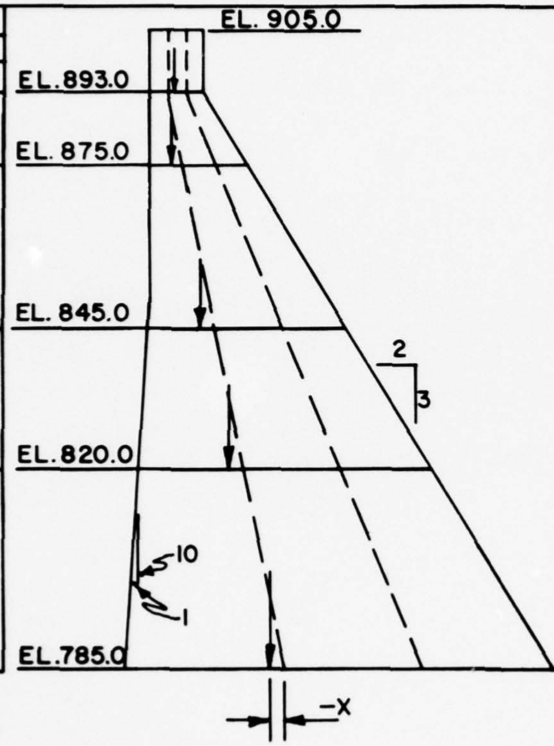
LEGEND

U - Uplift unit pressure (ksf)
ΣU - Total uplift on given plane (kips)
ΣH - Total of all horizontal forces above the given elevations (kips)
ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
s.f. - Shear friction factor
X - Distance inside middle third (+); outside (-); (ft)
Pi - Maximum pricipal stress at face of dam (ksf)
Pv - Vertical component of stress at face of dam (ksf)
Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN C

STABILITY ANALYSIS LOADING CONDITION V

UPSTREAM P"i	UPSTREAM P"v	UPSTREAM P"n	Elevation	DOWNSTREAM P'i	DOWNSTREAM P'v	DOWNSTREAM P'n	GENERAL ΣU	GENERAL ΣV	GENERAL ΣH	GENERAL f	GENERAL S.F.	GENERAL X
2.34	2.34	0.0	EL. 893.0	1.26	1.26	0.0	0.0	21.6	2.2	0.100	400.0	+1.3
6.75	6.75	0.0	EL. 875.0	0.0	0.0	0.0	0.0	70.3	7.0	0.100	215.0	-1.0
12.1	12.1	0.0	EL. 845.0	0.0	0.0	0.0	0.0	223.0	22.3	0.100	119.0	-2.4
14.83	14.68	0.0	EL. 820.0	0.0	0.0	0.0	0.0	425.0	42.5	0.100	98.0	-1.8
19.45	19.23	0.0	EL. 785.0	0.0	0.0	0.0	0.0	826.0	82.6	0.100	75.0	-1.3



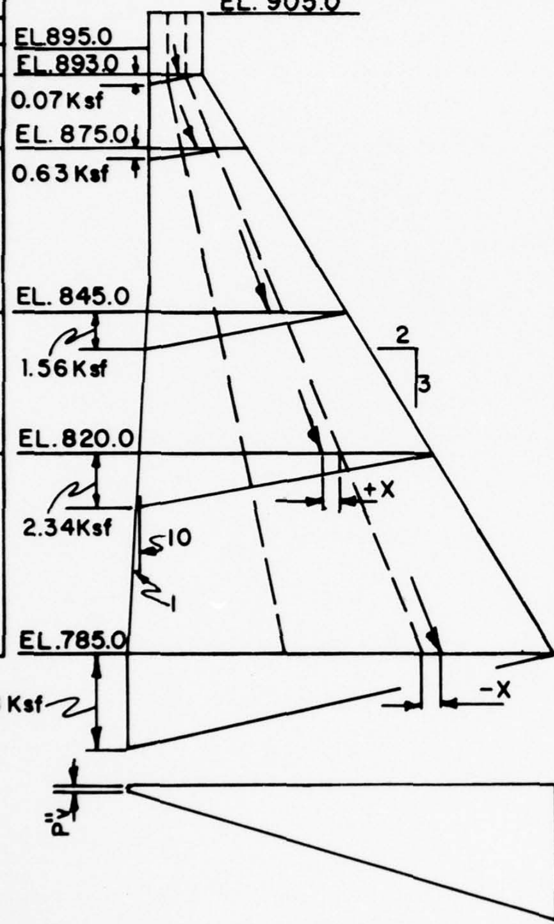
LEGEND

- U – Uplift unit pressure (ksf)
- ΣU – Total uplift on given plane (kips)
- ΣH – Total of all horizontal forces above the given elevations (kips)
- ΣV – Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f – Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. – Shear friction factor
- X – Distance inside middle third (+); outside (–); (ft)
- Pi – Maximum pricipal stress at face of dam (ksf)
- Pv – Vertical component of stress at face of dam (ksf)
- Pn – Normal component of pressure due to external load at face of dam (ksf)

PLAN C

STABILITY ANALYSIS LOADING CONDITION VI

	UPSTREAM				DOWNSTREAM			GENERAL					
Elevation	P''i	P''v	P''n		P'i	P'v	P'n	ΣU	ΣV	ΣH	f	S.F.	X
EL. 893.0	1.15	1.15	0.21		2.39	2.39	0.0	0.4	21.2	2.4	0.113	361.0	+1.3
EL. 875.0	2.94	2.94	1.49		3.32	2.29	0.0	7.5	62.8	22.7	0.362	76.0	+4.5
EL. 845.0	2.18	2.18	3.50		9.26	6.41	0.0	34.3	188.7	113.0	0.600	28.0	+3.8
EL. 820.0	1.28	1.27	5.14		14.70	10.08	0.0	73.7	361.1	241.7	0.670	18.8	+0.4
EL. 785.0	0.0	0.0	7.44		22.22	15.40	0.0	207.0	649.0	502.0	0.775	12.1	-2.0



LEGEND

- U - Uplift unit pressure (ksf)
- ΣU - Total uplift on given plane (kips)
- ΣH - Total of all horizontal forces above the given elevations (kips)
- ΣV - Total of vertical forces acting on a plane, including the effect of uplift (kips)
- f - Friction coefficient (All sliding resisted by friction) ΣH/ΣV
- s.f. - Shear friction factor
- X - Distance inside middle third (+); outside (-); (ft)
- Pi - Maximum pricipal stress at face of dam (ksf)
- Pv - Vertical component of stress at face of dam (ksf)
- Pn - Normal component of pressure due to external load at face of dam (ksf)

PLAN A & B

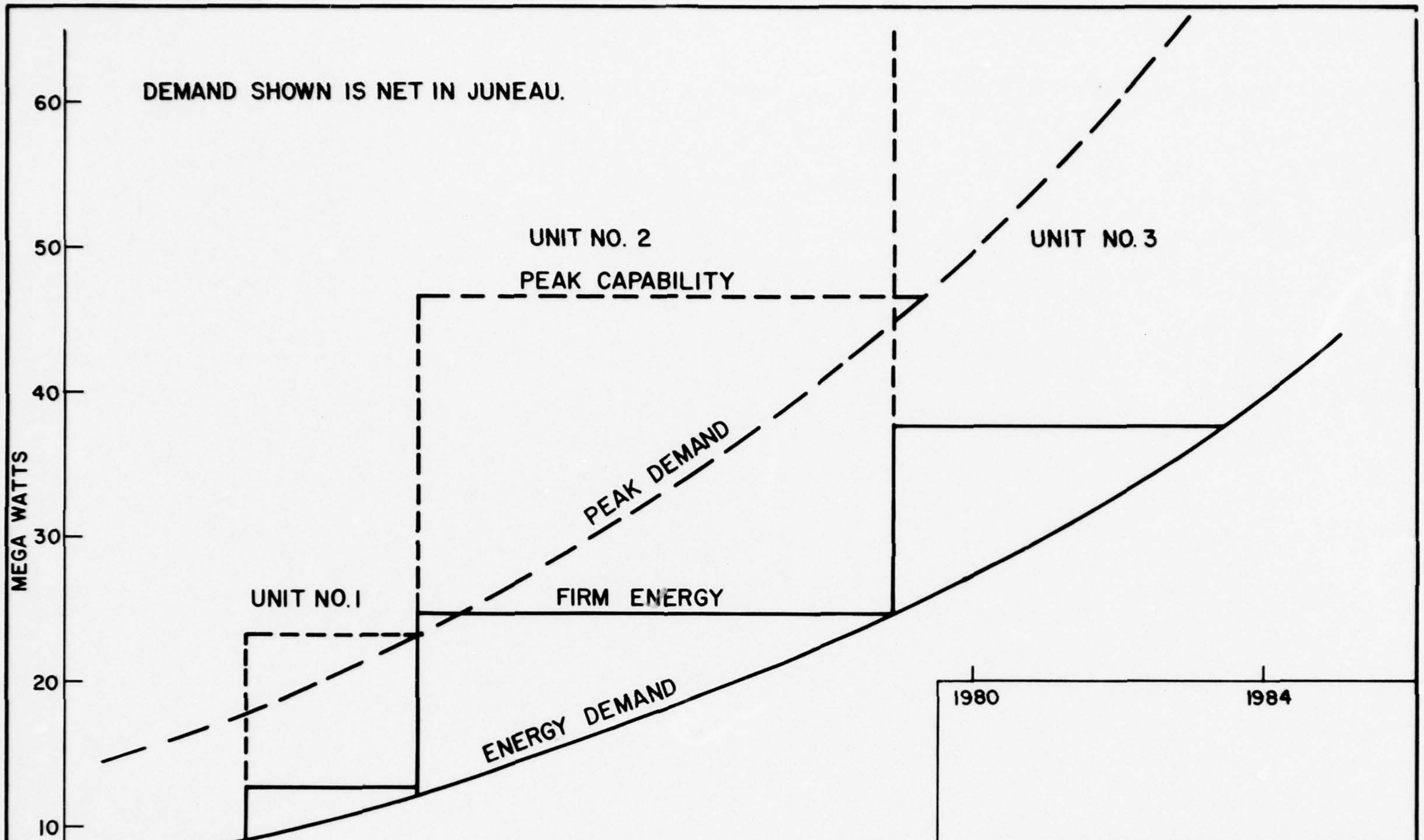
MAXIMUM STRESSES, SLIDING FACTORS, AND MINIMUM SHEAR-FRICTION FACTORS

NONOVERFLOW AND SPILLWAY SECTIONS

Loading Conditions:	Nonoverflow Section					Spillway Section				
	Stress lbs. per sq. in.			Max. sliding factor	Min. shear-friction factor	Stress lbs. per sq. in.			Max. sliding factor	Min. shear-friction factor
	Direct		Max. shear			Direct		Max. shear		
	Compr.	Tens.				Compr.	Tens			
A. Normal conditions:										
1. Reservoir empty	117.00	—	11.60	0.017	1800.0	119.00	—	5.90	0.011	2050
2. Normal full reservoir operation	135.30	—	62.50	0.623	16.0	127.00	-10.83	59.70	0.804	16.8
B. Including earthquake effect:										
1. Reservoir empty	135.00	—	13.35	0.100	75.0	133.50	—	6.70	0.100	77.8
2. Normal full reservoir operation	155.00	—	71.50	0.775	12.1	147.00	—	69.30	0.455	13.4
C. Flood conditions:	141.00	—	64.80	0.690	14.8	133.00	—	62.30	0.590	15.4

PLAN C
MAXIMUM STRESSES, SLIDING FACTORS, AND MINIMUM SHEAR-FRICTION FACTORS
NONOVERFLOW AND SPILLWAY SECTIONS

Loading Conditions:	Nonoverflow Section					Spillway Section				
	Stress lbs. per sq. in.			Max. sliding factor	Min. shear—friction factor	Stress lbs. per sq. in.			Max. sliding factor	Min. shear—friction factor
	Direct		Max. shear			Direct		Max. shear		
	Compr.	Tens.				Compr.	Tens.			
A. Normal conditions:										
1. Reservoir empty	117.00	—	11.60	0.017	1800.0	119.00	—	5.90	0.011	2050
2. Normal full reservoir operation	135.30	—	62.50	0.623	16.0	127.00	-10.83	59.70	0.804	16.8
B. Including earthquake effect										
1. Reservoir empty	135.00	—	13.35	0.100	75.0	133.50	—	6.70	0.100	77.8
2. Normal full reservoir operation	155.00	—	71.50	0.775	12.1	147.00	—	69.30	0.455	13.4
C. Flood conditions:	141.00	—	64.80	0.690	14.8	133.00	—	62.30	0.590	15.4



SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

LOAD VS CAPABILITY

COMPARISON

U. S. ARMY ENGINEER DISTRICT, ALASKA
PLANNING AND REPORTS BRANCH
PREPARED BY MDW DATE 18 OCT. 65

DESIGN MEMORANDUM NO. 7 FIGURE 19

DESIGN MEMORANDUM NO. 7 FIGURE 19

CORPS OF ENGINEERS

ITEM	D.M. NO.	CALENDAR YR 65 (J A S O N D) / FISCAL YR 66 (J A S O N D J F M A M J)	CALENDAR YR 66 / FISCAL YR 67 (J A S O N D J F M A M J)	CALENDAR YR 67 / FISCAL YR 68 (J A S O N D J F M A M J)
DESIGN MEMORANDUMS				
D.M. NO. 7: GENERAL DESIGN MEMORANDUM				
D.M. NO. 5: ACCESS & CONSTRUCTION FACILITIES				
D.M. NO. 14: PERMANENT OPERATING EQUIPMENT				
D.M. NO. 16: DIVERSION & OUTLET WORKS				
D.M. NO. 10: POWER INTAKE WORKS				
D.M. NO. 8; POWERHOUSE PRELIM. DESIGN REPORT				
D.M. NO. 17: TAILRACE & POWERHOUSE AREA GRADING				
D.M. NO. 13: MAIN DAM & SPILLWAY				
D.M. NO. 15: BUILDINGS, GROUNDS & UTILITIES				
D.M. NO. 9: TRANSMISSION FACILITIES				
D.M. NO. 11: REAL ESTATE				
D.M. NO. 12: GENERAL GEOLOGY				
PLANS & SPECS & CONTRACTS	D.M. NO.			
(1) PROCURE TURBINES	8			
(2) PROCURE BRIDGE CRANE	8			
(3) PROCURE OTHER MECH. & ELEC. EQUIPMENT	8			
(4) PROCURE GENERATORS	8			
(5) CONTRACT A9: ACCESS FAC., TUNNEL & POWERHOUSE EXCAV.				
A9-1: ACCESS & CONSTRUCTION FACILITIES	5			
A9-2: DIVERSION WORKS & EXCAV. OF OUTLET TUNNEL	16			
A9-3: EXCAVATION OF ACCESS ADIT & POWER TUNNEL	10			
A9-4: EXCAVATION OF PENSTOCK TUNNEL	10			
A9-5: EXCAVATION OF SURGE TANK SHAFT	10			
A9-6: EXCAVATION FOR POWERHOUSE	8 & 17			
A9-7: TAILRACE CHANNEL	17			
A9-8: POWERHOUSE AREA GRADING	17			
A9-9: DAM FOUNDATION GROUTING & PRESTRESSING	13			
(6) TRANSMISSION, LINE CLEARING	9			
(7) CONTRACT A9: DAM, TUNNEL LINING & POWERHOUSE				
B9-1: MAIN DAM & SPILLWAY	13			
B9-2: LINING OF OUTLET TUNNEL	13			
B9-3: POWER INTAKE STRUCTURE	10			
B9-4: LINING OF POWER TUNNEL	10			
B9-5: LINING OF PENSTOCK TUNNEL & ACCESS VAULT	10			
B9-6: LINING OF SURGE TANK	10			
B9-7: POWERHOUSE STRUCTURE & EQUIPT. INSTALLATION	8			
B9-8: SWITCHYARD	8 & 9			
B9-9: BUILDINGS, GROUNDS & UTILITIES	15			
(8) TRANSMISSION, SUBMARINE CABLE	9			
(9) PERMANENT OPERATING EQUIPMENT	14			
(10) TRANSMISSION, OVERHEAD LINE & JUNEAU SUBSTATION	9			

DESIGN MEMO — REVIEW & APPROVAL — PLANS & SPECS — REVIEW & APPROVAL — ADVERTISE & AWARD — SHOP TIME OR MOBILIZATION AND PREPARTORY WORK — INSTALLATION OR CONSTRUCTION

1497-66

SNETTISHAM
ALAS
FIRST STAGE
DESIGN & CON
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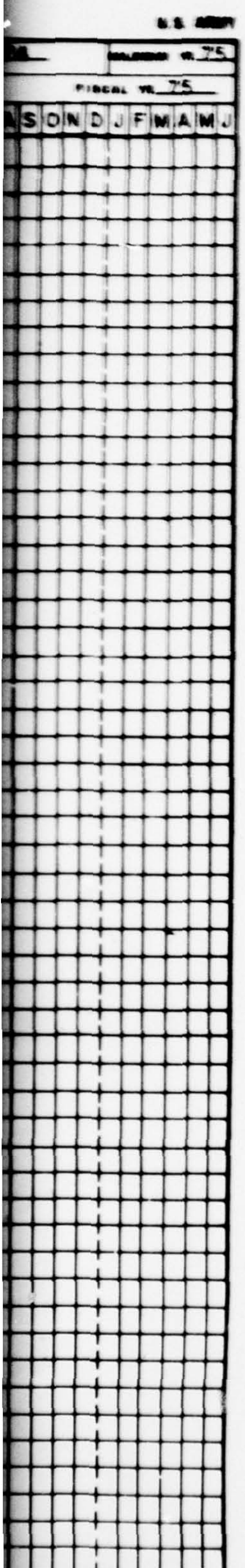
ALAS
PLANNING & RE
PREPARED BY W.D.L.

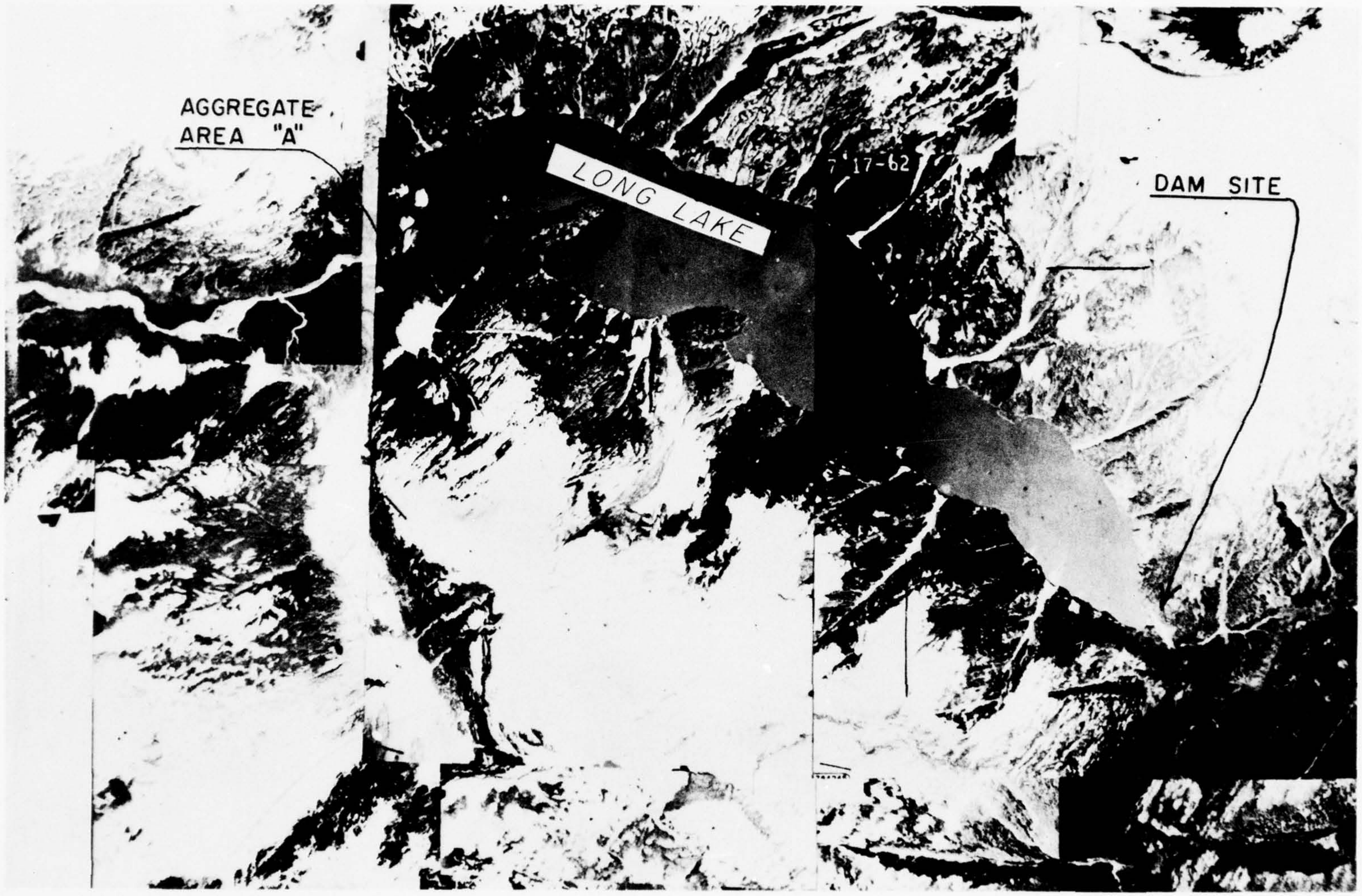
DESIGN MEMORANDUM NO. 7



HAM PROJECT,
ALASKA
GE DEVELOPMENT
CONSTRUCTION
CHEDULE .
ENGINEER DISTRICT,
ALASKA
REPORTS BRANCH
D.L. DATE: 5 NOV 65

O. 7, FIGURE 20 3

PHOTOS



SNETTISHAM PROJECT

SHOWING ROCK LINED LONG LAKE RESERVOIR

LONG LAKE RESERVOIR

PHOTO 1

SNETTISHAM PROJECT LONG LAKE DAM SITE

LONG LAKE DAM SITE AREA SHOWING ISOLATED ISLAND PORTION AND DOWNSTREAM TRACES OF LOW ANGLED JOINT PLANES

PHOTO 2

AD-A067 894 CORPS OF ENGINEERS ANCHORAGE AK ALASKA DISTRICT F/G 13/12
SNETTISHAM PROJECT ALASKA. FIRST STAGE DEVELOPMENT. DESIGN MEMO--ETC(U)
OCT 65

UNCLASSIFIED NL

3 OF 4

AD
A067894

SNETTISHAM PROJECT LONG LAKE RESERVOIR

SHOWING ROCK LINED BASIN FOR PRESENT LONG LAKE - FUTURE RESERVOIR WILL RAISE LAKE 80 FEET

PHOTO 3

SNETTISHAM PROJECT

LONG LAKE DAM SITE

LOOKING DOWN PRESENT LONG LAKE AT OUTLET RIDGE WHERE DAM SITE IS LOCATED

PHOTO 4

SNETTISHAM PROJECT

LONG LAKE DAM SITE

DAM SITE AS SEEN FROM LEFT ABUTMENT, SHOWNIG THE ACCELERATED DROP OFF OF ROCK SURFACE DOWNSTREAM WHICH, WHEN COMBINED WITH LOW ANGLED JOINTS ACROSS DAM SITE RIDGE, MAY CREATE A POTENTIAL SLIDING STABILITY PROBLEM.

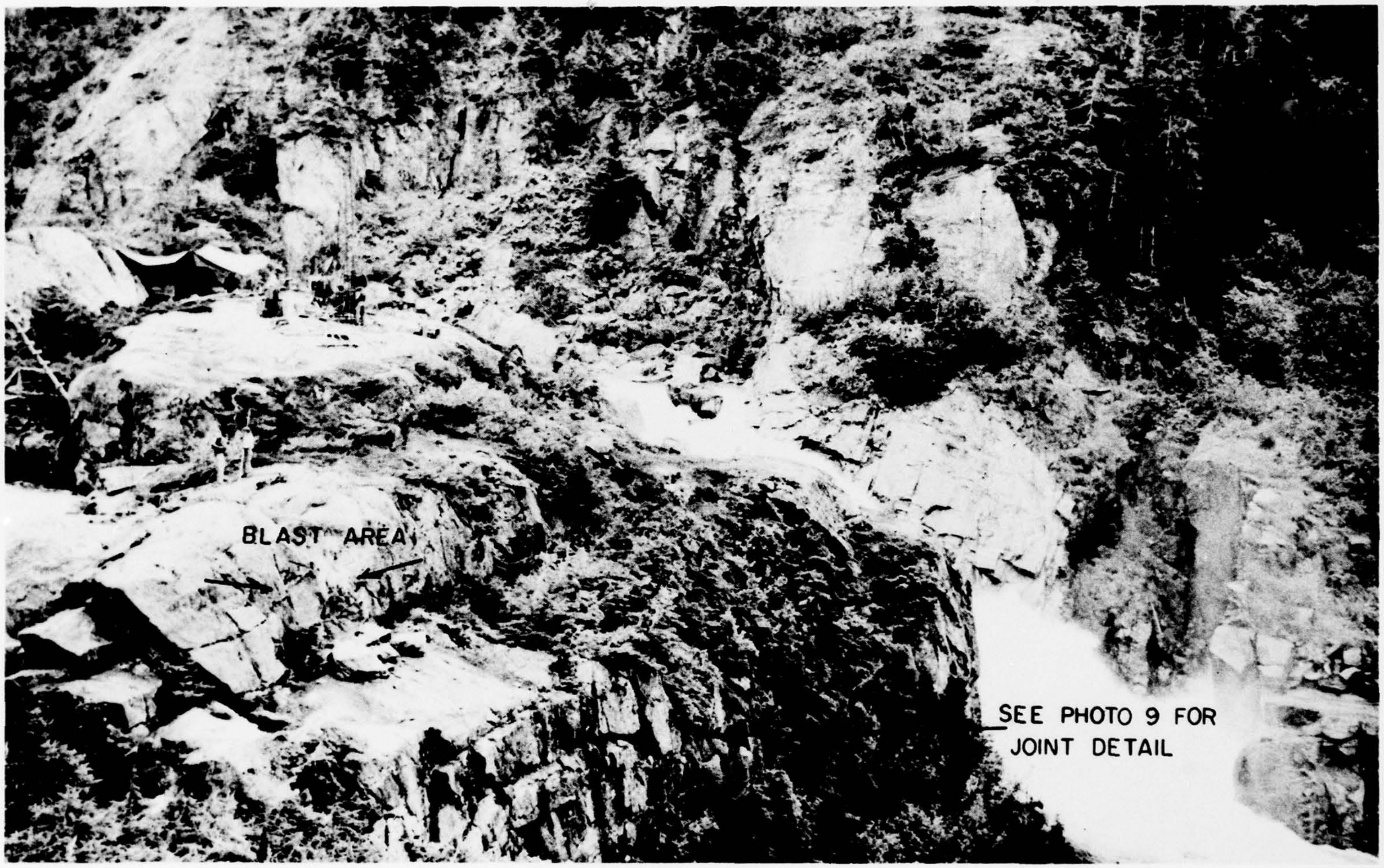
PHOTO 5

SNETTISHAM PROJECT

LONG LAKE DRILL BARGE

LONG LAKE DAM SITE

PHOTO 6



SNETTISHAM PROJECT

LONG LAKE DAM SITE

SHOWING ROCK BLASTED OUT OF ONE OF THE PROMINENT LOW ANGLED JOINTS TO GIVE FRESH EXPOSURE OF MATERIAL ON JOINT SURFACE. DRILL RIG AT LOCATION DH-49.

PHOTO 7

SNETTISHAM PROJECT

LONG LAKE DAM SITE

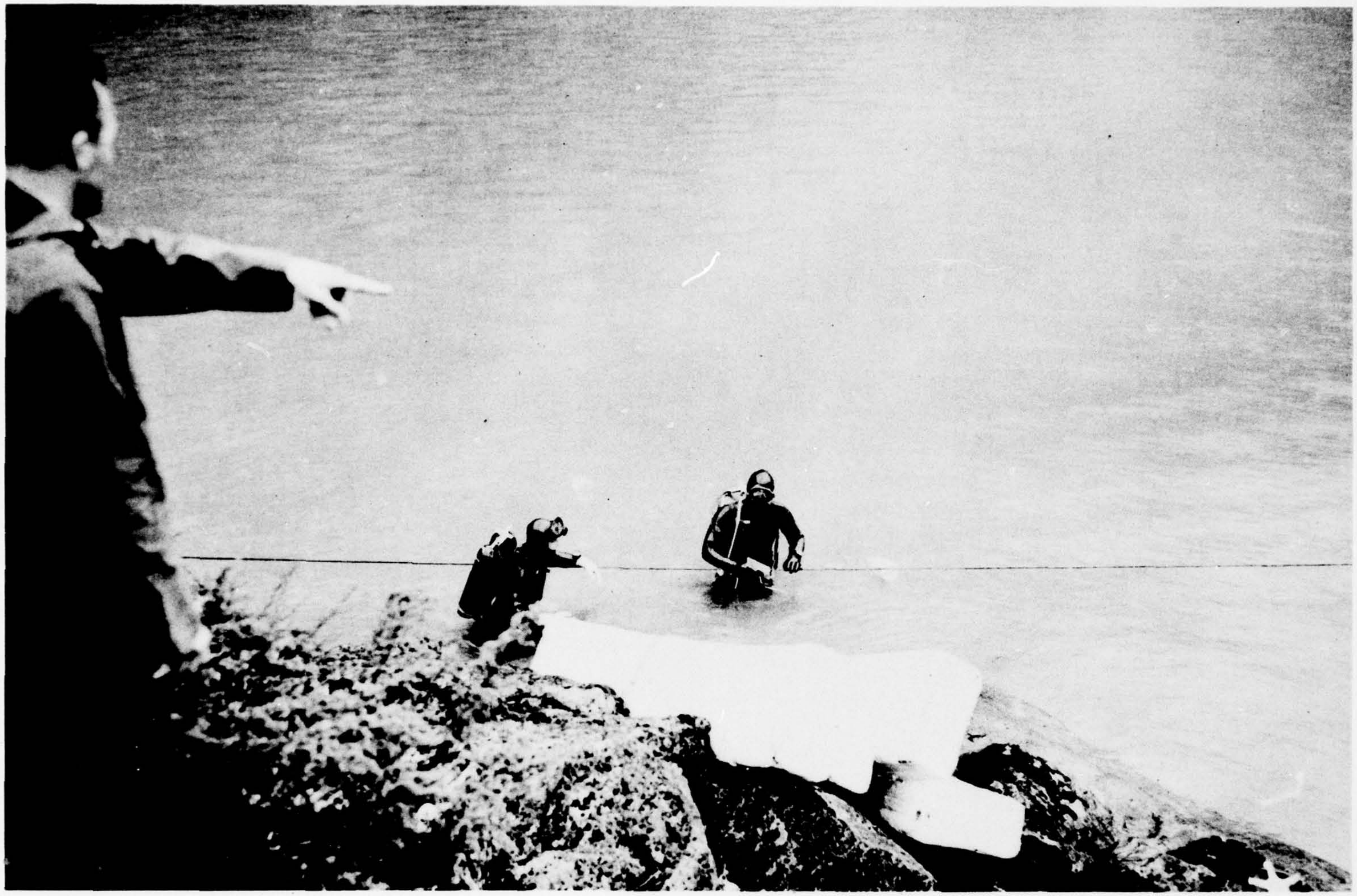
SHOWING CLOSE UP OF LOW ANGLE JOINT EXPOSURE (PHOTO 7 ABOVE). MATERIAL IMMEDIATELY SURROUNDING JOINT IS A DECOMPOSED GRANITE OR GRUS (TAN ϕ IS ESTIMATED AT 0.65).

PHOTO 8

SNETTISHAM PROJECT　　　LONG LAKE DAM SITE

SHOWING TRACES OF LOW-ANGLED JOINTS (ELEV. 785'±) OUTCROPPING ON DOWNSTREAM FACE OF THE DAM SITE RIDGE. SCUBA DIVER-GEOLOGISTS HAVE REPORTED SIMILAR LOW-ANGLED JOINT TRACES ON THE UPSTREAM UNDER-WATER FACE OF THE DAM SITE RIDGE. (SEE PHOTO 13) VIEW SHOWN IS IN ISLAND AREA OF DAM SITE.

PHOTO 9



SNETTISHAM PROJECT LONG LAKE DAM SITE

SHOWING SCUBA DIVER-GEOLOGIST TEAM FROM SCRIPPS INSTITUTE PREPARING FOR DIVE TRAVERSE OF LONG LAKE BOTTOM

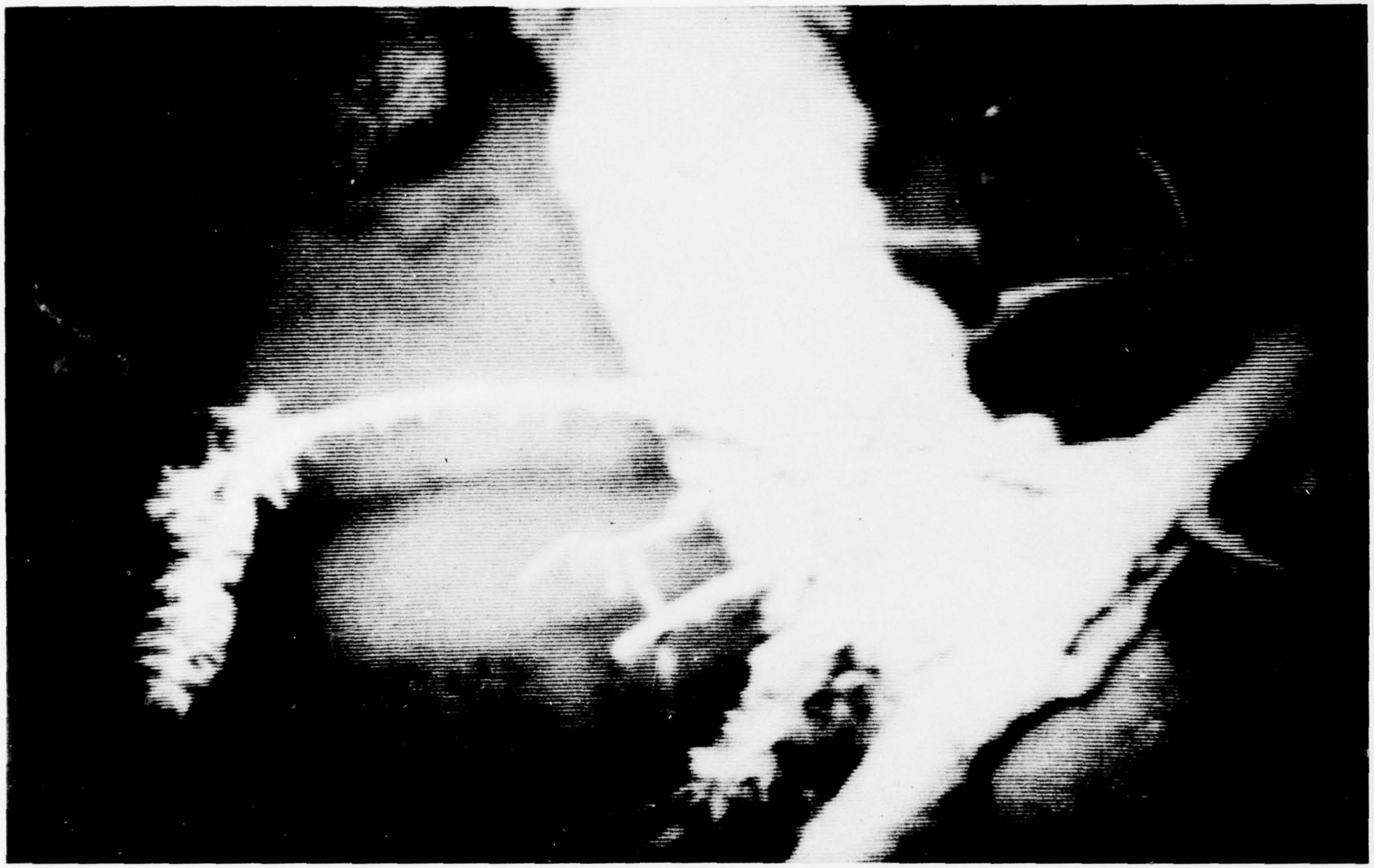
PHOTO 10

SNETTISHAM PROJECT

LONG LAKE DAM SITE

SHOWING UNDERWATER TV CAMERA WITH HIGH INTENSITY LIGHT ATTACHMENT

PHOTO 11



SNETTISHAM PROJECT

LONG LAKE DAM SITE

SHOWING **EXISTENCE** OF LARGE SIZE TREE DEBRIS ON LAKE BOTTOM IN **FRONT** OF DAM SITE RIDGE. THIS HERETOFORE UNKNOWN EXISTENCE OF TREE DEBRIS HAS CAUSED TRASH RACK PROVISIONS TO BE MADE FOR INTAKE GATES AND RESERVOIR OUTLET TUNNEL.

PHOTO 12

SNETTISHAM PROJECT

LONG LAKE DAM SITE

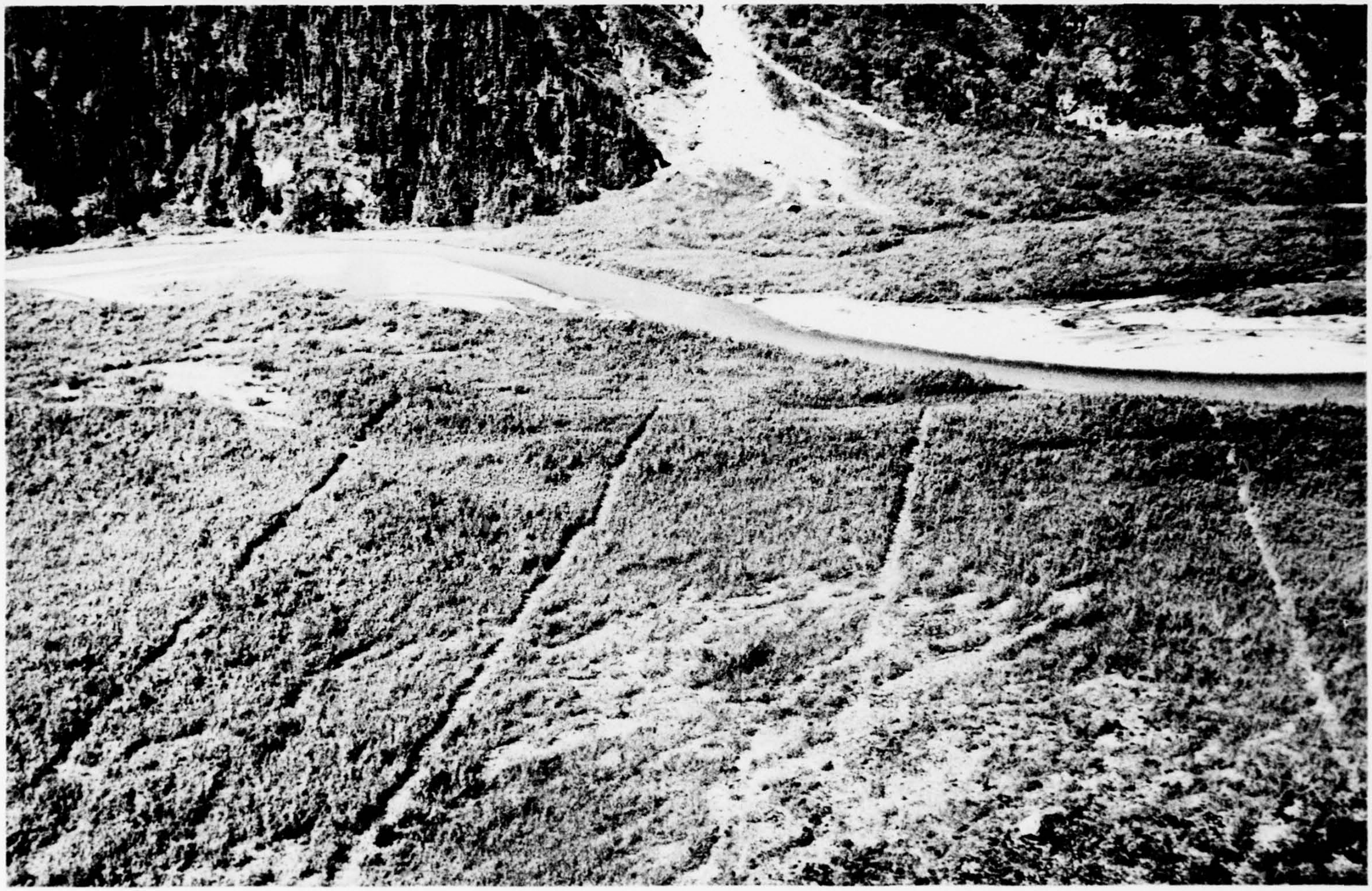
UNDERWATER TV SHOT OF LOW ANGLED JOINT TRACE OUTCROPPING ON UPSTREAM ROCK FACE OF DAM SITE

PHOTO 13

SNETTISHAM PROJECT — LONG LAKE DAM SITE

UNDERWATER TV SHOT SHOWING TREES AND BOULDERS ON LAKE BOTTOM APPROXIMATELY 100 FEET DEPTH JUST UP-STREAM OF SOUTH DAM SITE RIDGE.

PHOTO 14



SNETTISHAM PROJECT

CONCRETE AGGREGATE

CONCRETE AGGREGATE AREA "A" OUTWASH FAN AT HEAD OF LONG LAKE SHOWING BRUSHED LINES IN PREPARATION FOR DIGGING TEST PITS IN AN APPROXIMATE GRID PATTERN.

PHOTO 15

SNETTISHAM PROJECT

CONCRETE AGGREGATE

SHOWING GLACIER CREEK AGGREGATE DEPOSIT AREA "B" WITH TEMPORARY GOVERNMENT CAMP IN BACKGROUND.

PHOTO 16

SNETTISHAM PROJECT

CONCRETE AGGREGATE

SHOWING GLACIER CREEK AGGREGATE DEPOSIT AREA "B" NEAR MOUTH OF GLACIER CREEK.

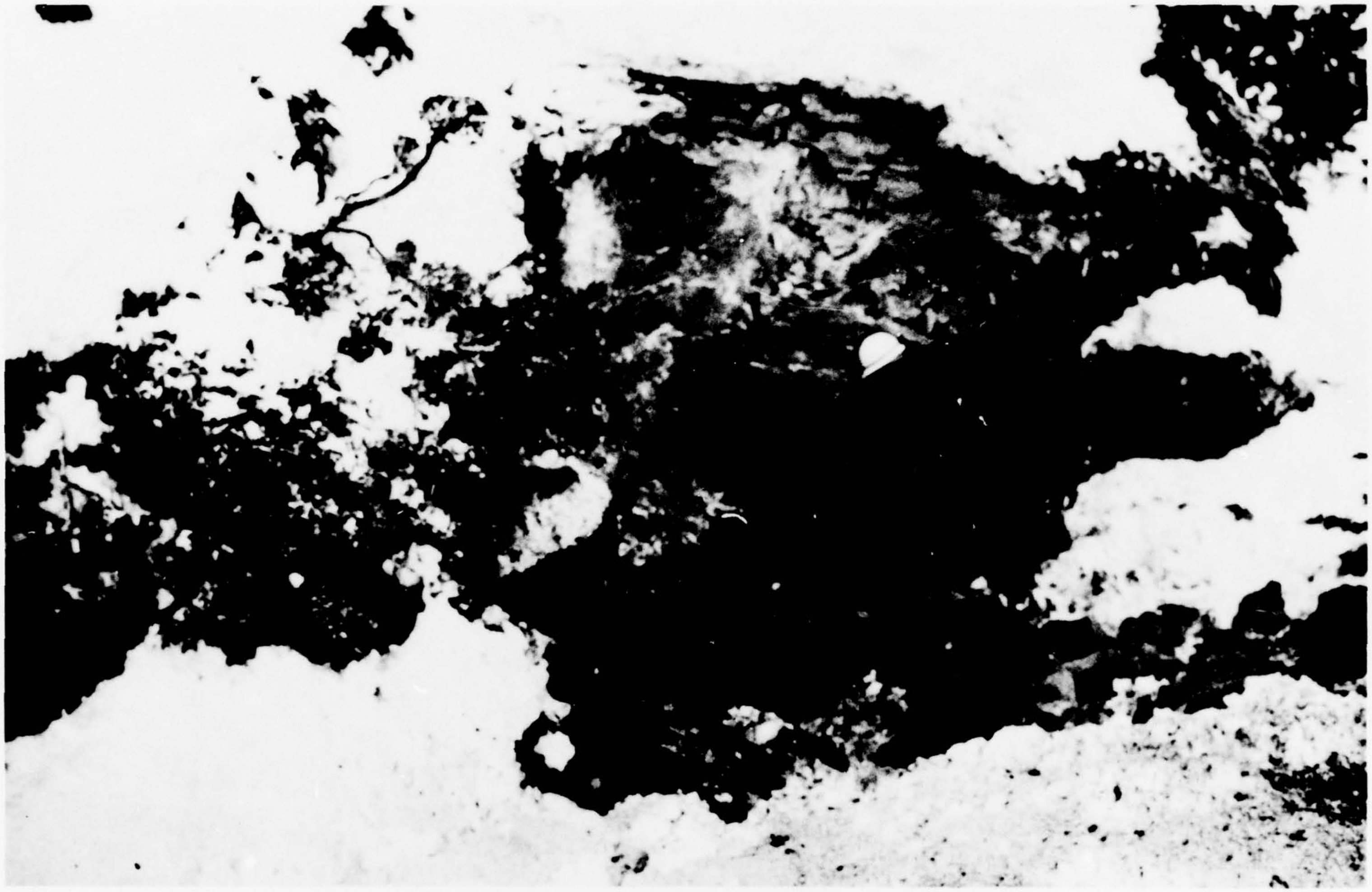
PHOTO 17

SNETTISHAM PROJECT

CONCRETE AGGREGATE

SHOWING CONCRETE AGGREGATE AREA "C" CONSISTING OF RELATIVE FRESH AND UNWEATHERED OUTCROPS OF QUARTZ DIORITE LEDGE ROCK IN A NATURAL TERRACED AREA JUST OFF THE LEFT ABUTMENT OF DAM SITE. 9 NOV 1964.

PHOTO 18



SNETTISHAM PROJECT

CONCRETE AGGREGATE

CONCRETE AGGREGATE AREA "C" AFTER BLASTING-PREPARING TO TAKE BLASTED SAMPLES FOR SHIPMENT TO NPD TESTING LABORATORY FOR TRIAL MIX DESIGN OF MANUFACTURED AGGREGATE.

PHOTO 19

SNETTISHAM PROJECT POWER TUNNEL

GLACIER CREEK DRILL RIG, DH 22, DRILLING TO INTERSECT GLACIER CREEK FAULT-FAULT ZONE AT DEPTH SHOWED LESS PLASTIC FAULT GOUGE AND HIGHLY WEATHERED ROCK MATERIALS THAN WERE MAPPED IN SURFACE EXPOSURES OUTCROPPING AT CREEK LEVEL TO RIGHT OF PICTURE.

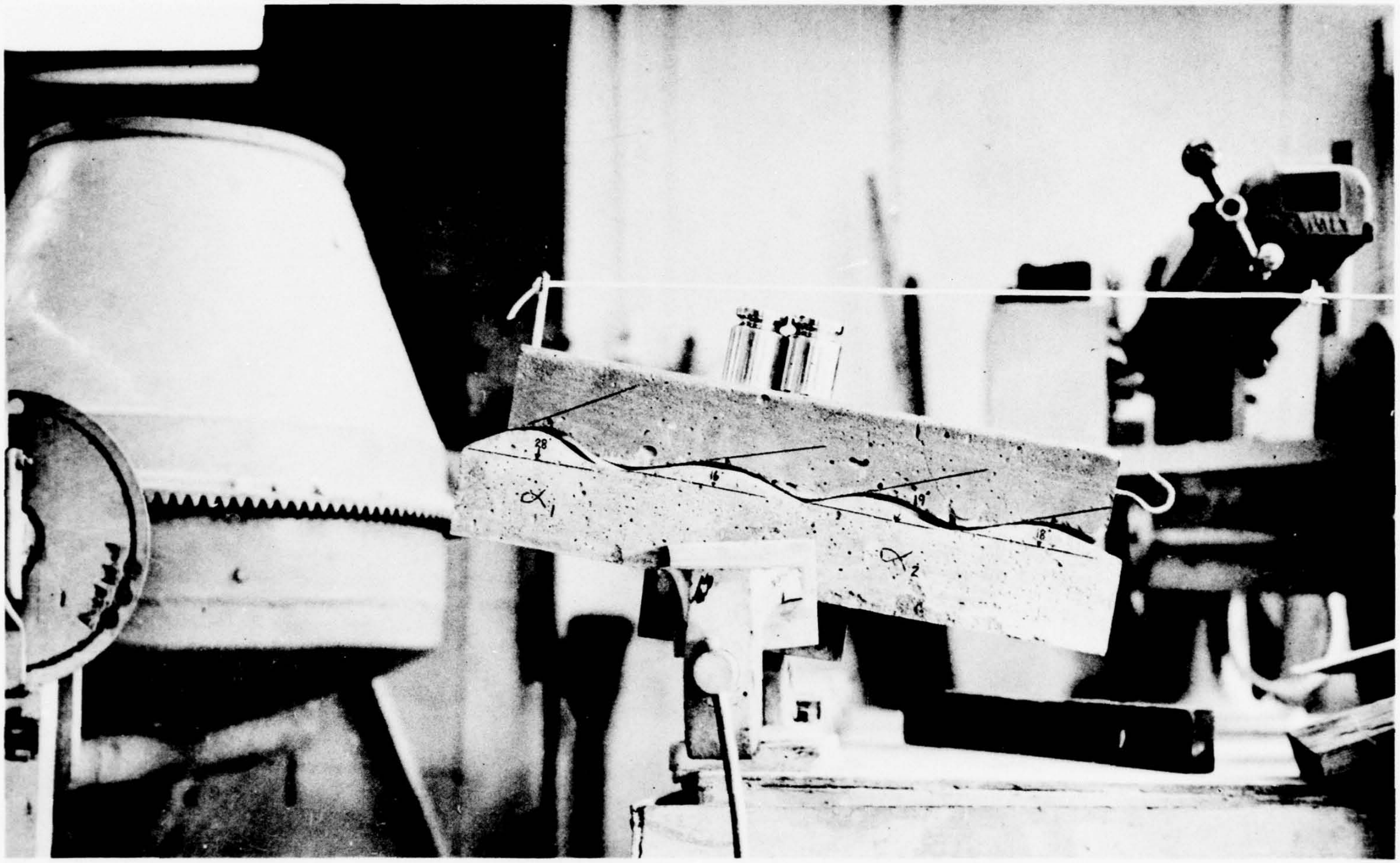
PHOTO 20

SNETTISHAM PROJECT

SLIDING FRICTION

ANALYSIS OF SLIDING ON UNDULATING LOW ANGLED JOINT SURFACE - SETUP WITH 10° "DOWNSTREAM" DIP TO BLOCK. (θ IS NEGATIVE). NOTE: WEIGHT (FORCE) REQUIRED TO MOVE UPPER BLOCK IS ACTING IN HORIZONTAL DIRECTION.

PHOTO 21

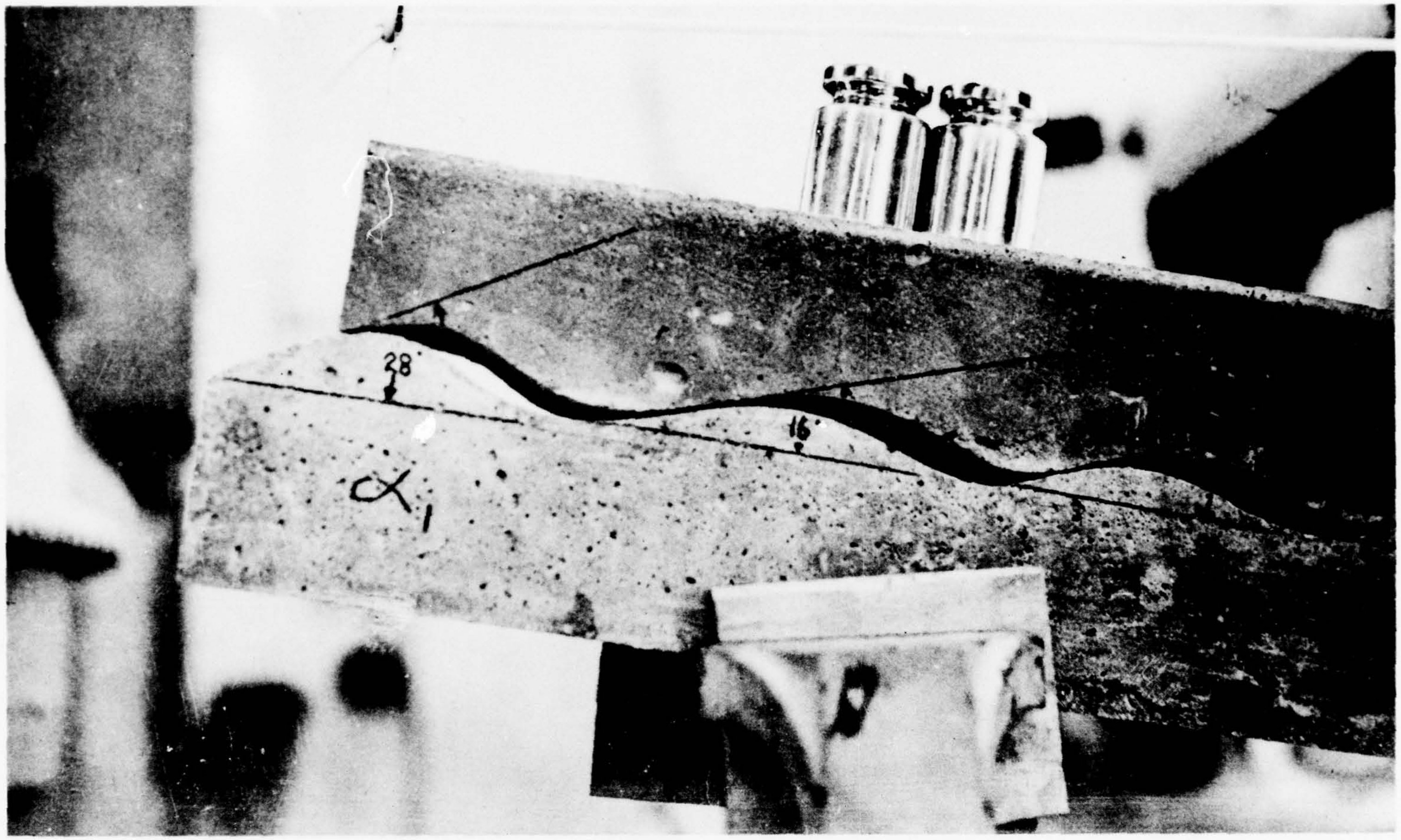


SNETTISHAM PROJECT

SLIDING FRICTION

ANALYSIS OF SLIDING ON UNDULATING SURFACE- SETUP WITH 10° "DOWNSTREAM" DIP TO BLOCK, (θ IS NEGATIVE). PHOTO SHOWS FIRST MOVEMENT. NOTE: OPEN AREAS OF NON CONTACT BETWEEN BLOCKS EXCEPT ON STEEPEST UNDULATION SLOPES (28°) AND (19°). "BUILT IN" ANGLES IN BLOCK ARE APPROXIMATIONS OF DAM SITE UNDULATION ANGLES ON LOW ANGLED JOINTS AS DETERMINED FROM BORE HOLE PHOTOGRAPHS.

PHOTO 22



SNETTISHAM PROJECT

SLIDING FRICTION

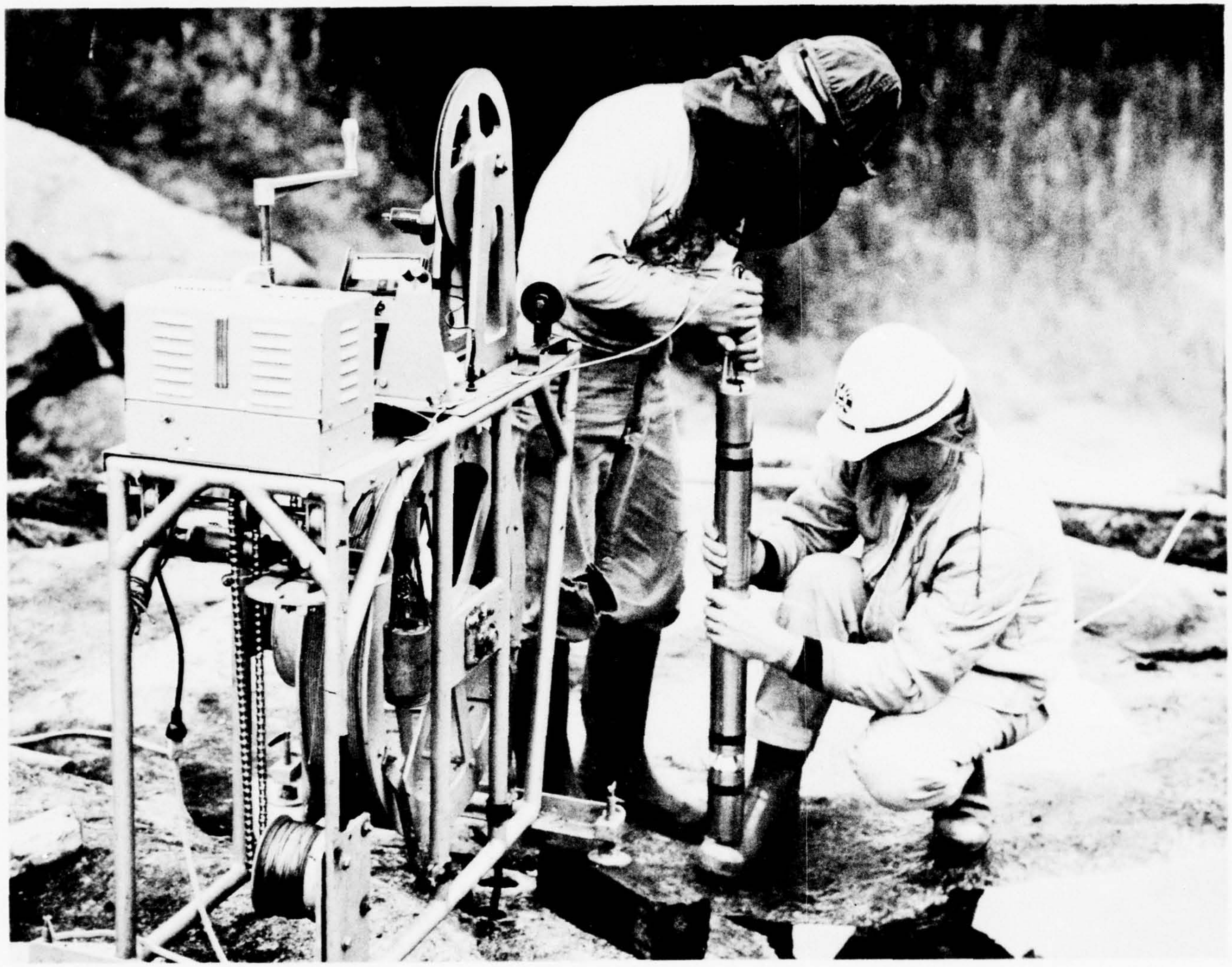
ANALYSIS OF SLIDING ON UNDULATING SURFACE - 10 "DOWNSTREAM" DIP TO BLOCK, (θ IS NEGATIVE). CLOSE UP OF MODEL SHOWING HOW WEIGHT IS SUPPORTED ON TWO STEEPEST UNDULATIONS (28° AND 19°) WITH NO CONTACT ON 16° UNDULATION. OF PARTICULAR INTEREST WAS THE RELATIONSHIP OF HORIZONTAL TO VERTICAL FORCES ($\Sigma H/\Sigma V$) REQUIRED TO JUST START MOVEMENT UNDER VARYING CONDITIONS OF ECCENTRIC LOADING AND CHANGING ANGULAR ATTITUDE OF BLOCK.

PHOTO 23

General Rock Types   Snettisham Project, Alaska

NX core samples showing degree of foliation, grain size and dark mineral content of varietal types. Left to Right: Sample Nos. DH 6-1, DH 8-3, DH 6-5, DH 10-4, DH 7-7, DH 7-1 & DH 6-2. Second, third & fourth samples from the left represent majority of material submitted.

PHOTO 24



BORE HOLE PHOTOGRAPHY BEING CONDUCTED AT DAM SITE PHOTO 25

EXHIBITS

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
ALASKA DISTRICT HEADQUARTERS
P. O. BOX 2567, JUNEAU, ALASKA 99801

IN REPLY REFER TO: 620

February 11, 1965

Your reference:
NPAEN-PR-R
NPAEN-PR-P

AIRMAIL

District Engineer
U.S. Army Engineer District, Alaska
P.O. Box 7002
Anchorage, Alaska 99501

Dear Sir:

The following are our comments by item as requested in your December 1, 1964 letter, reference No. NPAEN-PR-R:

Paragraph a. Permanent access to the dam should be provided for wheeled vehicles. If the contractor provides a road for construction purposes, it should be sufficient for our use. The high maintenance factor associated with a tramway should preclude any thought of its use for operational purposes.

Paragraph b. If a dam is to be constructed, an effort should be made to construct one in which gates are not required for the spillway. The high maintenance costs of gates and the lack of need for controlled spilling of water make them undesirable.

Paragraph c. There is no need for a vehicular bridge over either type of spillway, but an adequate footbridge should be provided.

Paragraph d. A low-level outlet in the dam should be provided. If the tunnel intake is below the bottom of the dam, facilities to draw the lake below the tunnel intake should be substituted for the low-level outlet in the dam.

Paragraph a. (Page 2) A single slot for the stoplogs and trash racks would create many additional operating problems of stacking and handling. In addition there is the serious

EXHIBIT 1.1

possibility of trash being dislodged when the trash racks are removed and thereby preventing proper seating of the stoplogs. We therefore recommend a separate downstream slot for the stoplogs. We concur with the plan to make access to the tunnel available only from an adit near the surge tank. We assume that an air vent will be provided at the lake end of the tunnel.

Paragraph b. (Page 2) It is our wish that two penstocks be provided, one for each lake. These penstocks should be interconnected near the units and valved in such a manner that either main penstock may be deactivated for repair or inspection without causing the outage of more than one unit in the entire plant.

Paragraph c. (Page 2) Regarding our requirements for housing for personnel, we still have this subject under study. We hope to be able to provide you some definite information on this in the near future.

The shop and storage space provided should be planned with the thought in mind that the Snettisham plant will be isolated from all means of transportation for days at a time. Also, since the project will be the supplier of practically all power in the area, we recommend a very complete shop with ample storage space for spare parts over and above that usually provided.

In reference to your December 1, 1964 letter, reference No. NPAEN-PR-P, we are furnishing the following comments:

Paragraph 1 and 2 - You may be assured that sufficient study went into the choosing of the proper line voltage and conductor size for the transmission line to best meet anticipated requirements. One of the specific responsibilities of the Bureau of Reclamation, as outlined in the exchange of letters between Assistant Secretary Holum and Mr. Califano in September 1963, is the determination of line voltage and conductor size.

Paragraph 3 - Our plan for development of a Southeast Alaska Transmission Grid does not envision further upgrading of voltage of the Snettisham-Juneau transmission line.

Paragraph 4 - There has been some concern regarding the reliability of a single circuit line from Snettisham. However, considering the potential reliability problem versus

EXHIBIT 1.2

cost involved for a second circuit, it is our belief that only a single, well planned and designed circuit should be installed initially with particular attention being given to tower location and protection in known slide areas and danger trees.

As requested in the last paragraph of your letter there is enclosed one reproducible print of Drawings 864-D-1560, -1562, and -1566, showing plan-profiles with structure types and locations used on a section of the Curecanti-Midway 230-kv Transmission Line. Also enclosed is an Abstract of Bids received for this line under Specifications DC-6013. These drawings, together with Specifications DC-6013, furnished with our letter of November 10, 1964 and data furnished with our letter of November 13, 1964, should provide the principal details for the section of line requiring steel tower construction. If there are specific details or applications on which you wish to have our recommendations, please advise.

We appreciate the opportunity to comment on these matters and hope that the necessary time taken to review these points has not caused delay of your design memoranda.

Sincerely yours,

Geo. N. Pierce

George N. Pierce
District Manager

Enclosures

EXHIBIT 1.3

FEDERAL POWER COMMISSION
REGIONAL OFFICE
555 BATTERY STREET, ROOM 415
SAN FRANCISCO 11, CALIF.

February 25, 1964

AIR MAIL

Colonel K. T. Sawyer
District Engineer
U. S. Army Engineer District, Alaska
Corps of Engineers
P. O. Box 7002
Anchorage, Alaska 99501

Dear Colonel Sawyer:

This is in answer to your letter of January 31, 1964 (your file reference NPAEN-PR-TP) requesting comments on specific questions which concern your study of the Snettisham Project.

a. The amount of reserve capacity required to maintain firm service would be equal to the largest reduction in total system dependable capacity which might result from an outage of a generating unit or transmission circuit at the time of the system peak.

At the present time the power supply in the Juneau area consists of small hydroelectric and internal-combustion plant generating units. The largest unit is 3,500 kilowatts. Another unit or a group of units, in reserve, equal in capacity to this unit would permit loads to be served on a firm basis. No reserve capacity would be needed if some loads could be dropped until the largest unit is returned to service, in the event of a forced outage.

With the addition of generating capacity such as may be installed at the Snettisham Project, firm service to the loads would be maintained by providing standby capacity in other power plants in the Juneau area equal to the dependable capacity of the largest project unit, or the capacity of its largest transmission circuit. This assumes that the greatest reduction in the supply to the Juneau area would be due to the loss of a Snettisham unit or line. On the other hand, it is conceivable that a thermal plant might be built or other hydro installations may be constructed with units larger than installed at Snettisham.

EXHIBIT 2.1

b. No amount of project power would be firm at the Juneau market if delivery is to be by one transmission circuit.

c. The alternative source of power to the project is considered by us at present to be a privately financed steam-electric plant located at Juneau tidewater. If power can be made dependably available from outlying system sources at a lower market cost, this cost would be a measure of the value of project power at market.

The value of hydroelectric power at Juneau based on cost of steam-electric power estimated in 1960 was $65.11 per kilowatt-year for dependable capacity plus 6.91 mills per kilowatt-hour for energy. The total unit value of power for a 70 percent load factor delivery, amounts to 17.53 mills per kilowatt-hour. This value was based on financing of a 49,500-kilowatt steam-electric plant with cost of money at 7 percent and with taxes of 6.77 percent of the investment. The cost for fuel was based on bunker oil at Juneau delivered for $3.20 per barrel.

By our letter of February 14, 1964, you were furnished at-steam plant values for the southwest area of Alaska for various plant sizes. These were based on cost of producing power at a 3 percent rate of interest and with no taxes. If the alternative plant capacity is to be initially comparable in size to the Snettisham installation, the value (because of the high cost of smaller units and other factors than were used for developing estimates of values furnished) would be somewhat greater.

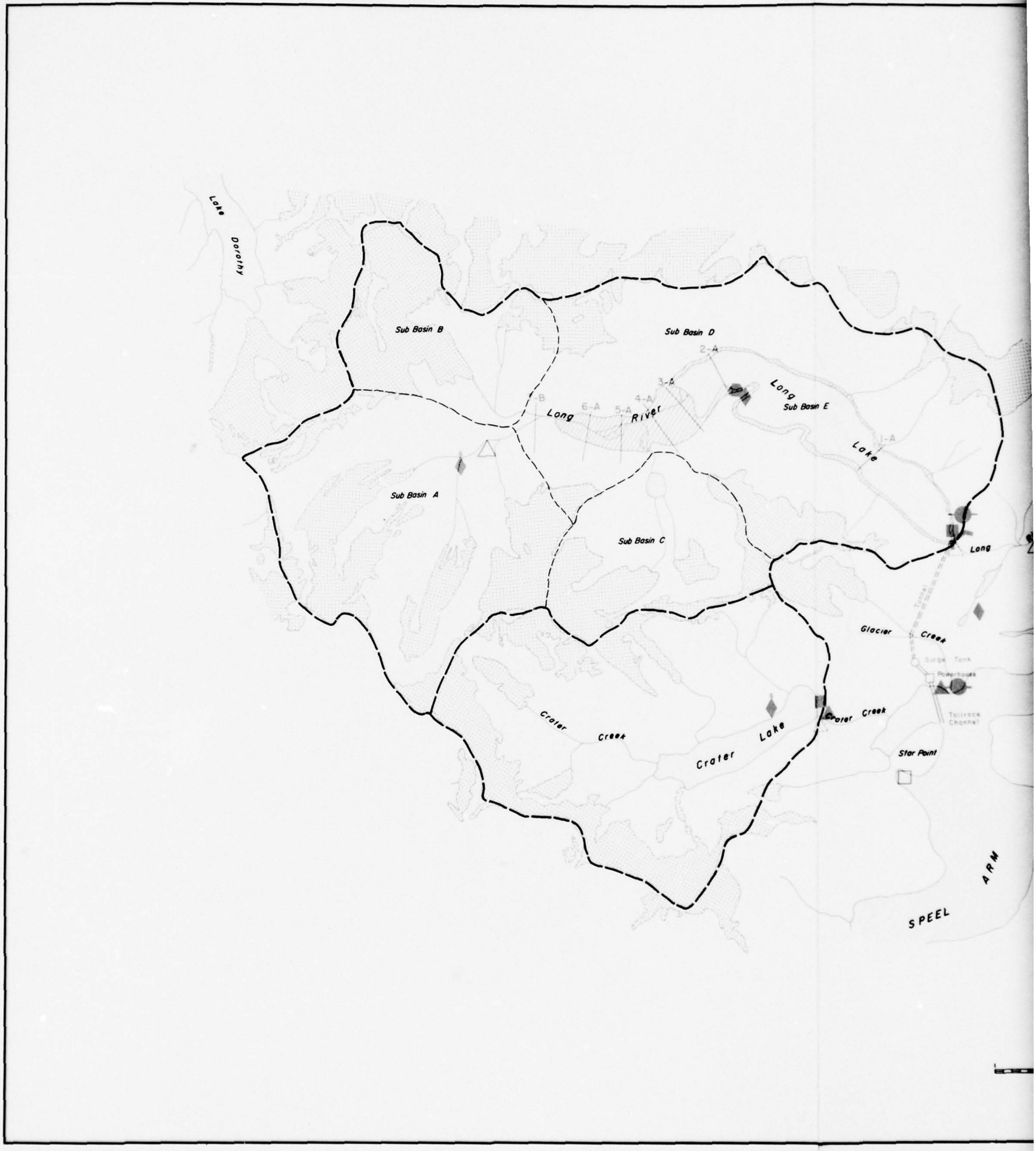
Sincerely yours,

M. Boyd Austin

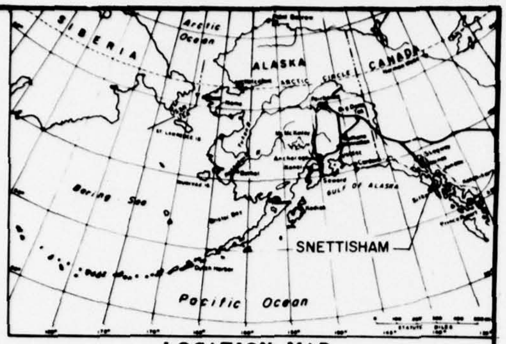
M. Boyd Austin
Regional Engineer

EXHIBIT 2.2

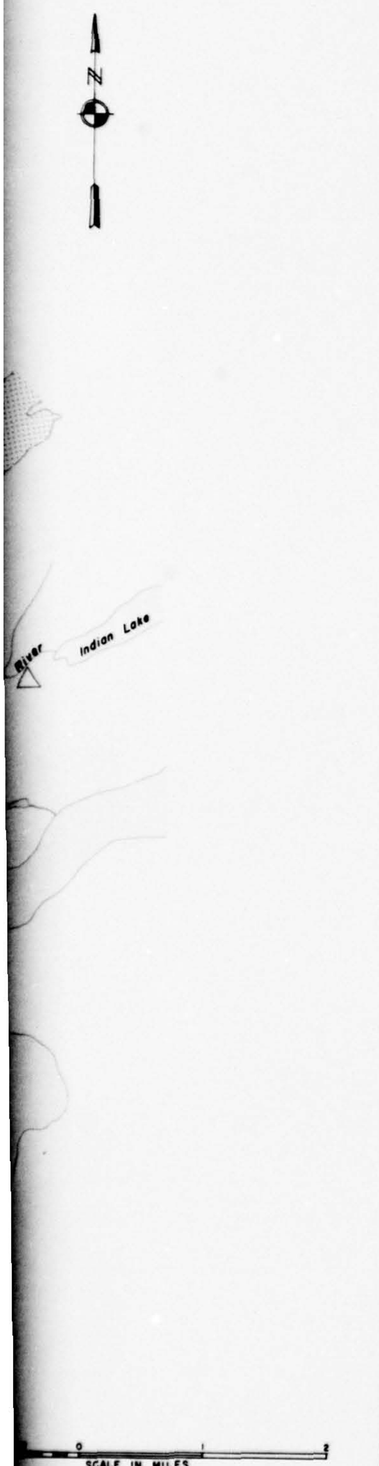
PLATES


Lake Dorothy
Sub Basin B
Sub Basin D
Sub Basin E
Sub Basin A
Sub Basin C
Long River
Long Lake
Long
Glacier Creek
Surge Tank
Powerhouse
Tailrace Channel
Crater Creek
Crater Lake
Star Point
SPEEL ARM

U. S. ARMY



LOCATION MAP



LEGEND

- PROPOSED PRECIPITATION, TEMPERATURE & WIND STATION
- EXISTING STREAM GAGING STATION
- DISCONTINUED STREAM GAGING STATION
- PROPOSED STREAM GAGING STATION
- EXISTING SNOW COURSE
- EXISTING AERIAL SNOW MARKER
- PROPOSED SNOW PRESSURE PILLOW
- PROPOSED LAKE STAGE RECORDER
- EXISITING TIDE STAGE RECORDER
- 1-A EXISTING SEDIMENTATION RANGES
- DRAINAGE DIVIDE
- SUB BASIN DIVIDE
- GLACIERS OR PERMANENT SNOW FIELDS
- FULL POOL ELEVATION, 895
- DAM

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: C.G.G.
DRAWN: E.M.L.
CHECKED: G.N.M.
PREPARED:

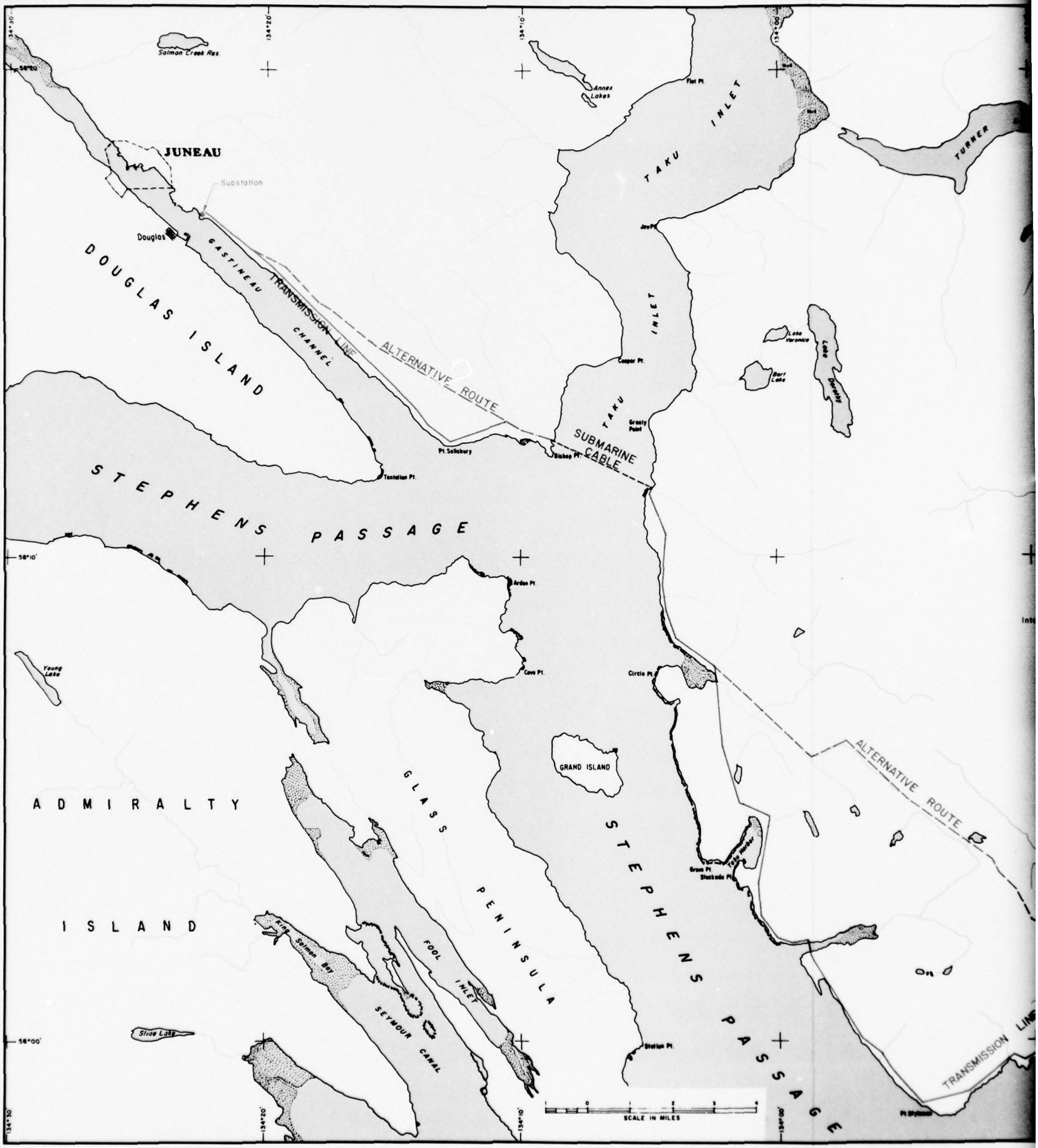
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
BASIN MAP

SUPERVISED: Walter W. Duncan — CHIEF, HYDROLOGY SECTION
SUBMITTED: [signature] — CHIEF, PLANNING & REPORTS BRANCH
RECOMMENDED: Warren George — CHIEF, ENGINEERING DIVISION
APPROVED: Chas. T. Tarley — COLONEL, CE, DISTRICT ENGINEER

SCALE AS SHOW — DATE 8 OCT 65
FILE NUMBER Q-4-5-113

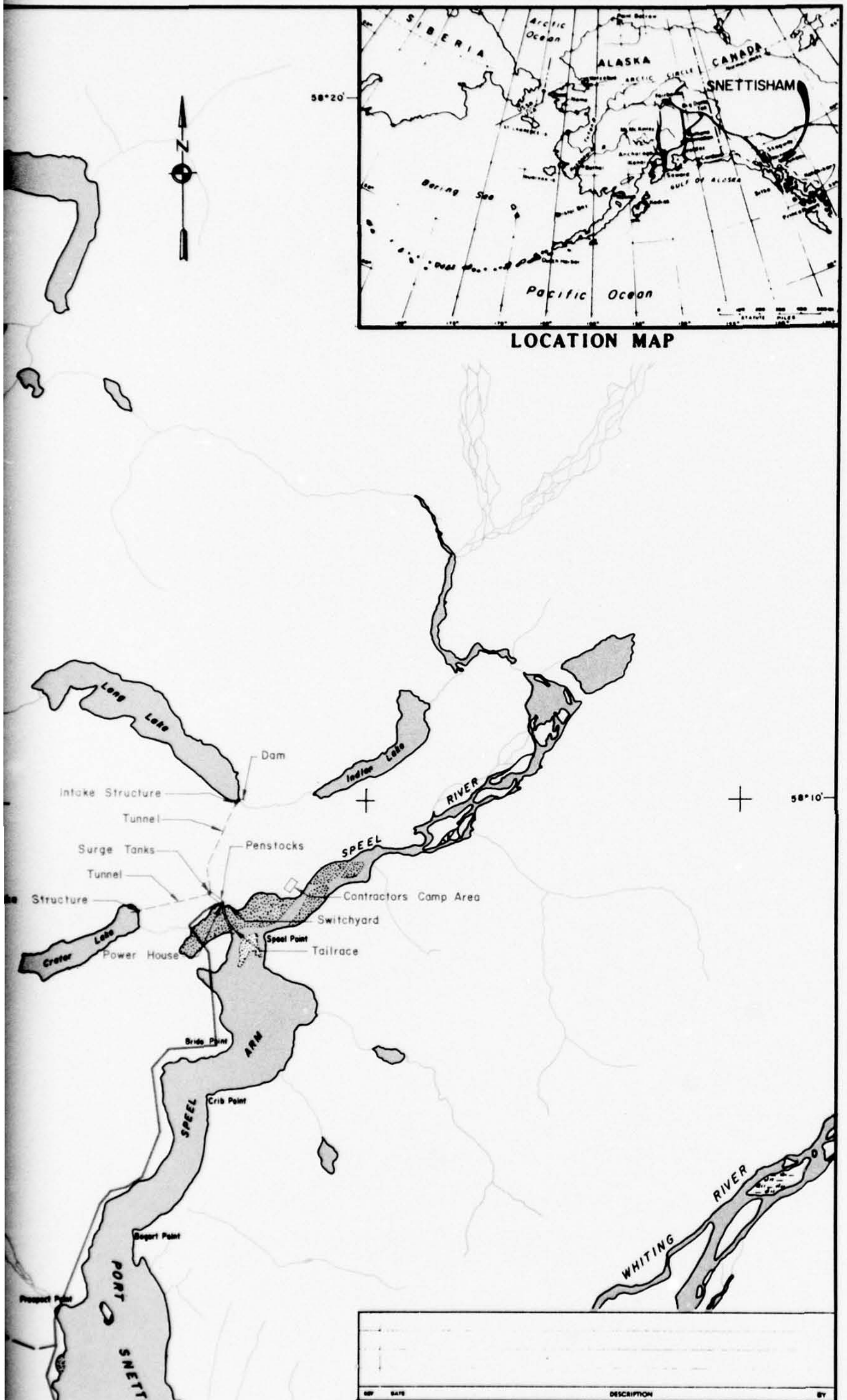
DESIGN MEMORANDUM NO. 7 PLATE NO. 2

2



JUNEAU
Substation
Douglas
Salmon Creek Res.
Annex Lakes
DOUGLAS ISLAND
GASTINEAU CHANNEL
TRANSMISSION LINE
ALTERNATIVE ROUTE
SUBMARINE CABLE
TAKU INLET
TURNER
Flat Pt.
Jaw Pt.
Cooper Pt.
Greely Point
Bishop Pt.
Pt. Salisbury
Lake Veronica
Bart Lake
Lake Dorothy
STEPHENS PASSAGE
Arden Pt.
Cove Pt.
Circle Pt.
GRAND ISLAND
Young Lake
ADMIRALTY ISLAND
GLASS PENINSULA
FOOL INLET
SEYMOUR CANAL
King Salmon Bay
Slide Lake
Station Pt.
Grave Pt.
Stockade Pt.
Taku Harbor
Pt. Styleman
ALTERNATIVE ROUTE
TRANSMISSION LINE
SCALE IN MILES
134°30'
134°20'
134°10'
134°00'
58°20'
58°10'
58°00'

U. S. ARMY



REV	DATE	DESCRIPTION	BY

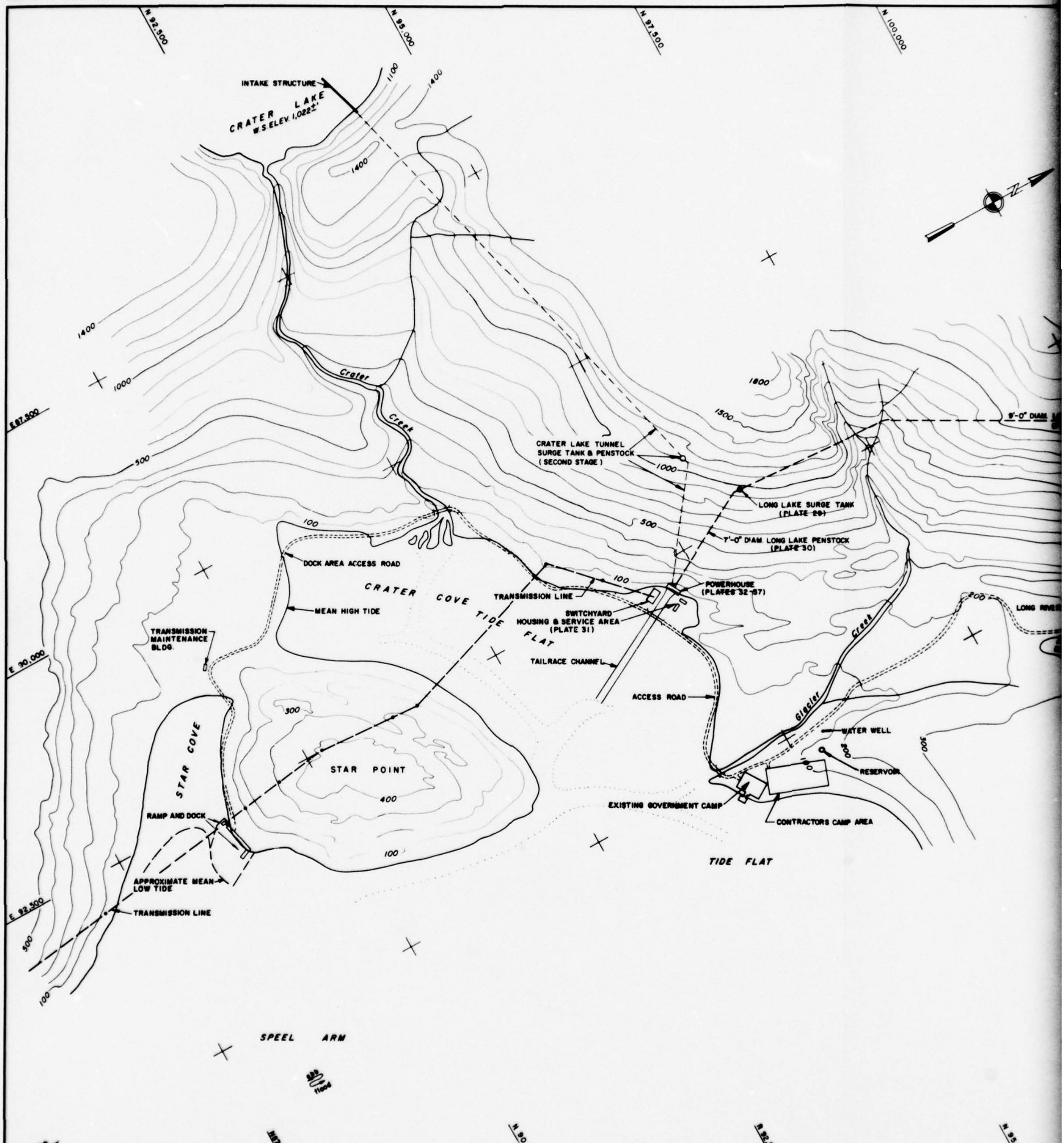
U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED:	SNETTISHAM PROJECT, ALASKA
DRAWN: E.A. & W.D.L.	FIRST STAGE DEVELOPMENT
TRACED: R.M.D. & P.G.	**GENERAL AREA**
CHECKED: W.D.L.	
PREPARED: W.D. Lloyd — CHIEF, DAM & SITES UNIT	
SUPERVISED: Earl J. Groenwald — CHIEF, INVESTIGATIONS & REPORTS SECTION	APPROVED: Geo. I. Farley — COLONEL, CE, DISTRICT ENGINEER
SUBMITTED: — CHIEF, PLANNING & REPORTS BRANCH	SCALE: AS SHOWN — DATE: 8 OCT 65
RECOMMENDED: Warren George — CHIEF, ENGINEERING DIVISION	FILE NUMBER: Q-4-5-113

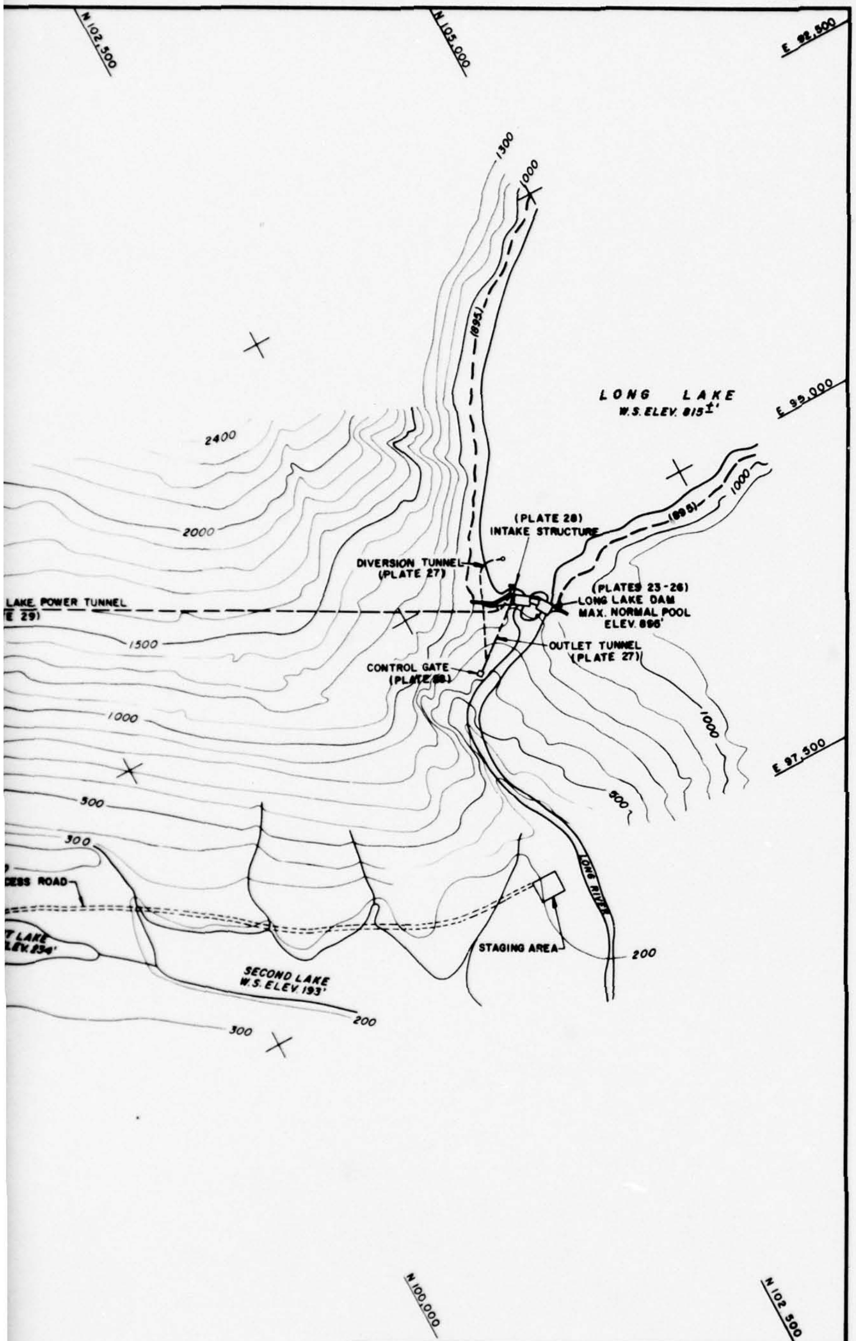
DESIGN MEMORANDUM NO. 7 PLATE NO. 1

2

CORPS OF ENGINEERS



U. S. ARMY



REVISION	DATE	DESCRIPTION

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

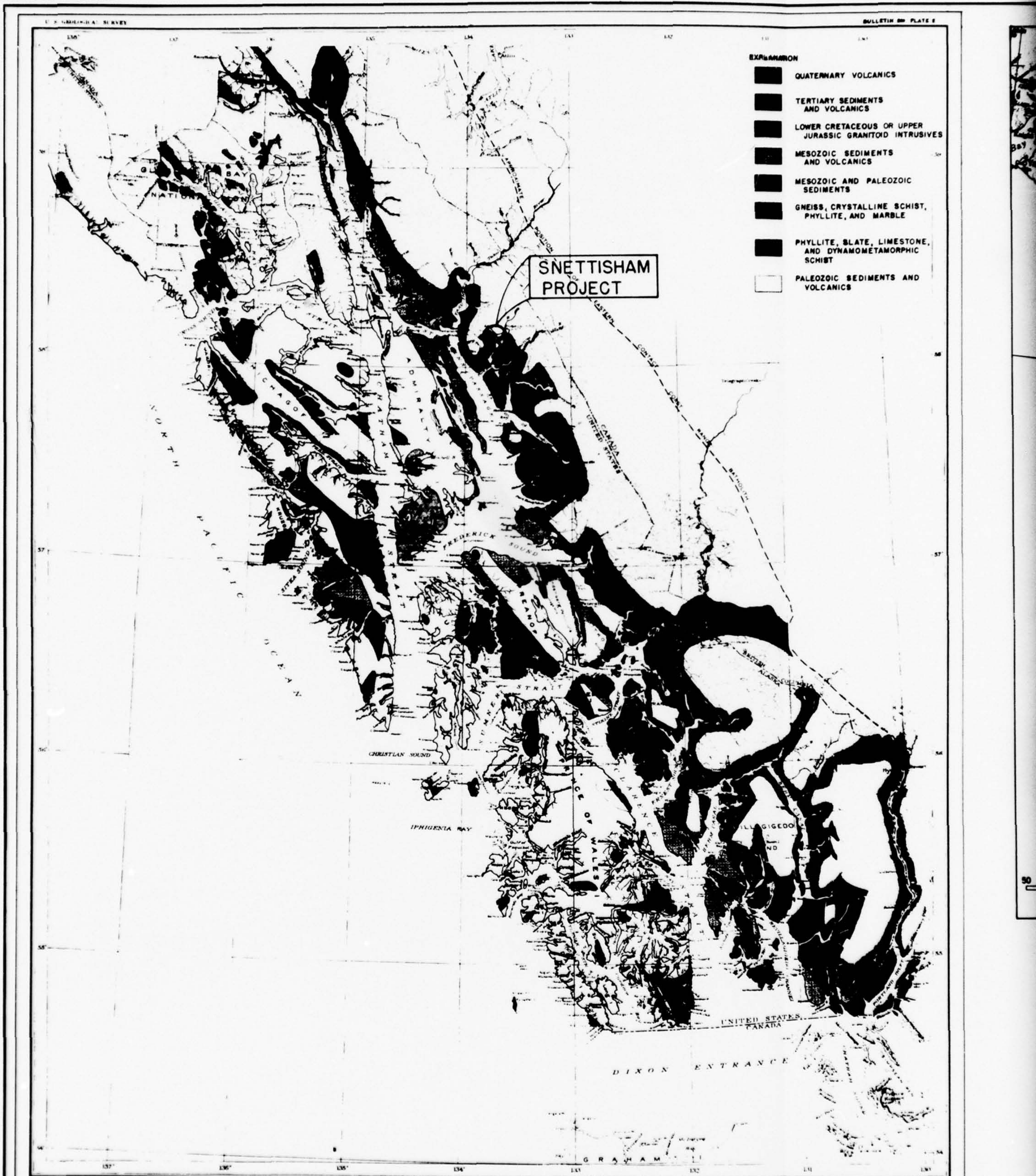
DESIGNED: W.D.L.
DRAWN: E.L.
CHECKED: W.D.L.
PREPARED: CHIEF, DAM & RES. UNIT
SUPERVISED: CHIEF, INVESTIGATIONS & REPORTS SECTION
SUBMITTED: CHIEF, PLANNING & REPORTS BRANCH
RECOMMENDED: CHIEF, ENGINEERING DIVISION

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

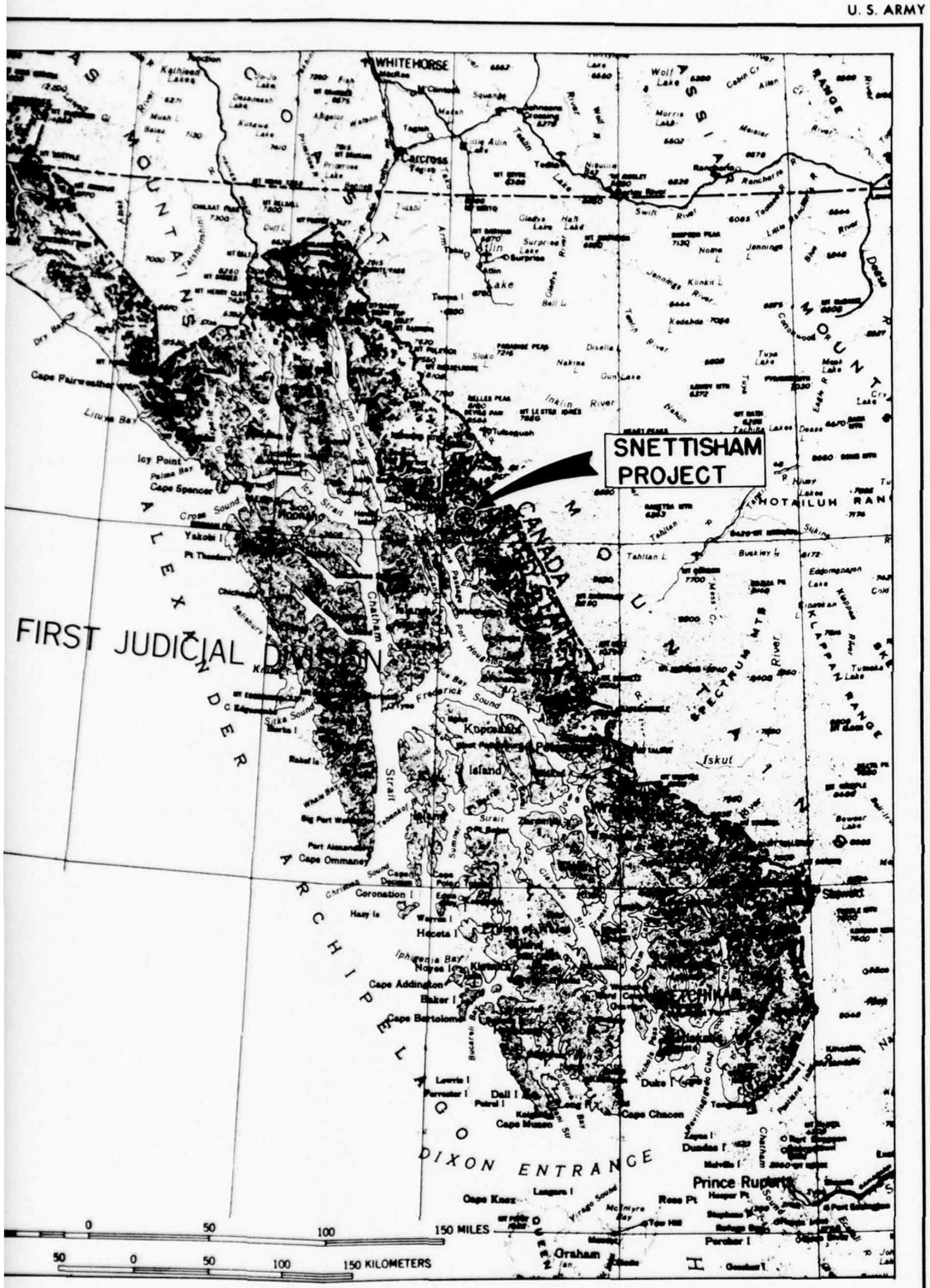
PROJECT PLAN

APPROVED: COLONEL, CE, DISTRICT ENGINEER
SCALE AS SHOWN
DATE 21 OCT 1966
FILE NUMBER Q-4-5-113
SHEET OF

2



GENERALIZED GEOLOGIC MAP OF SOUTHEASTERN ALASKA



REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

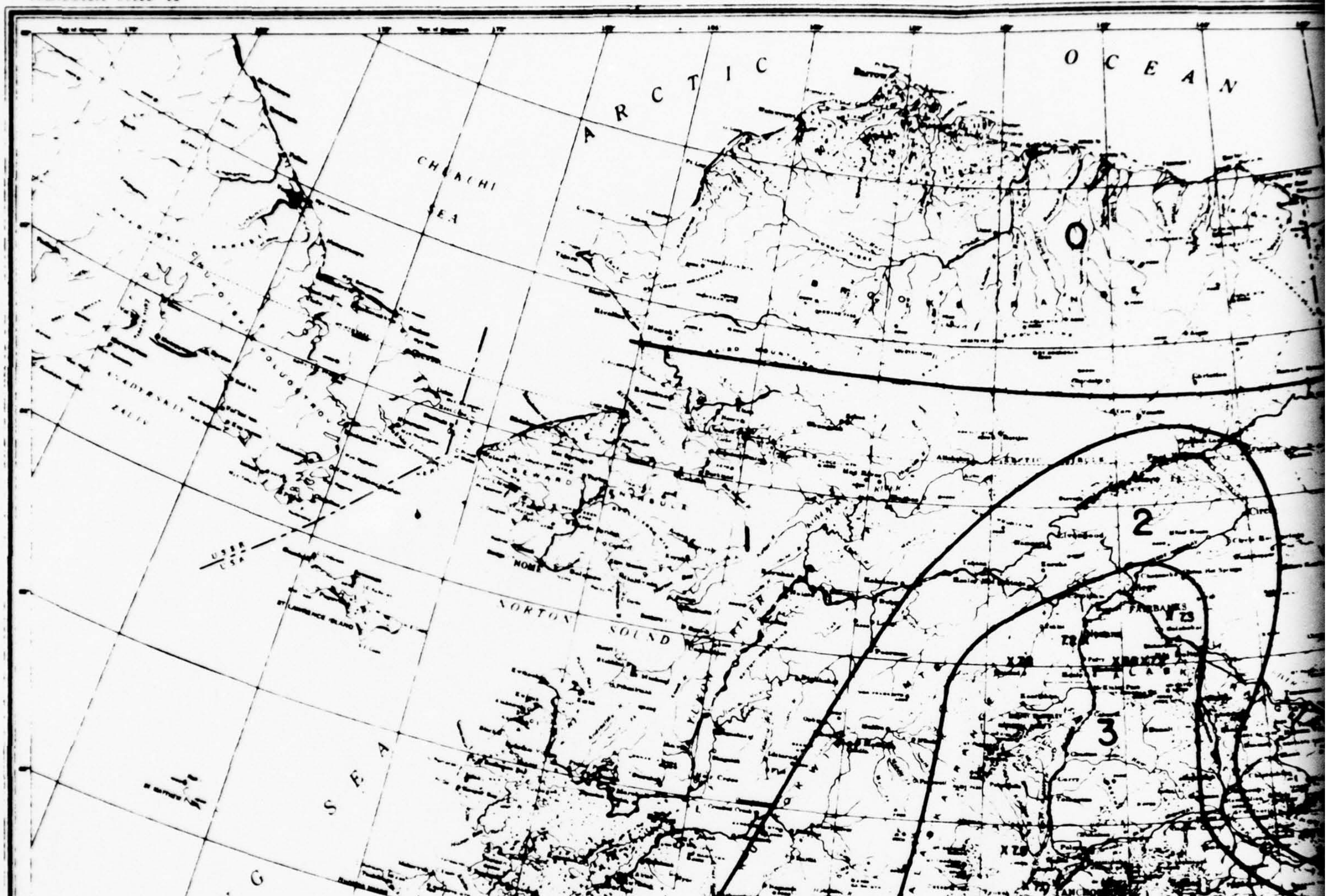
DESIGNED:
DRAWN:
CHECKED: D. J. LACHEL
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
GEOLOGIC & TOPOGRAPHIC
MAPS OF SOUTHEASTERN ALASKA

SUPERVISED: N. A. Dixon
CHIEF, GEOLOGY SECTION
SUBMITTED:
CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED:
CHIEF, ENGINEERING DIVISION

APPROVED:
COLONEL, CE, DISTRICT ENGINEER
SCALE
DATE 20 Oct 65
FILE NUMBER
SHEET OF

ALASKA MAP A


ARCTIC
OCEAN
CHUKCHI SEA
BERING SEA
NORTON SOUND
BRISTOL BAY
GULF OF A
PACIFIC
ALEUTIAN ISLANDS
NOME
FAIRBANKS
0
2
2
3
3
3
3

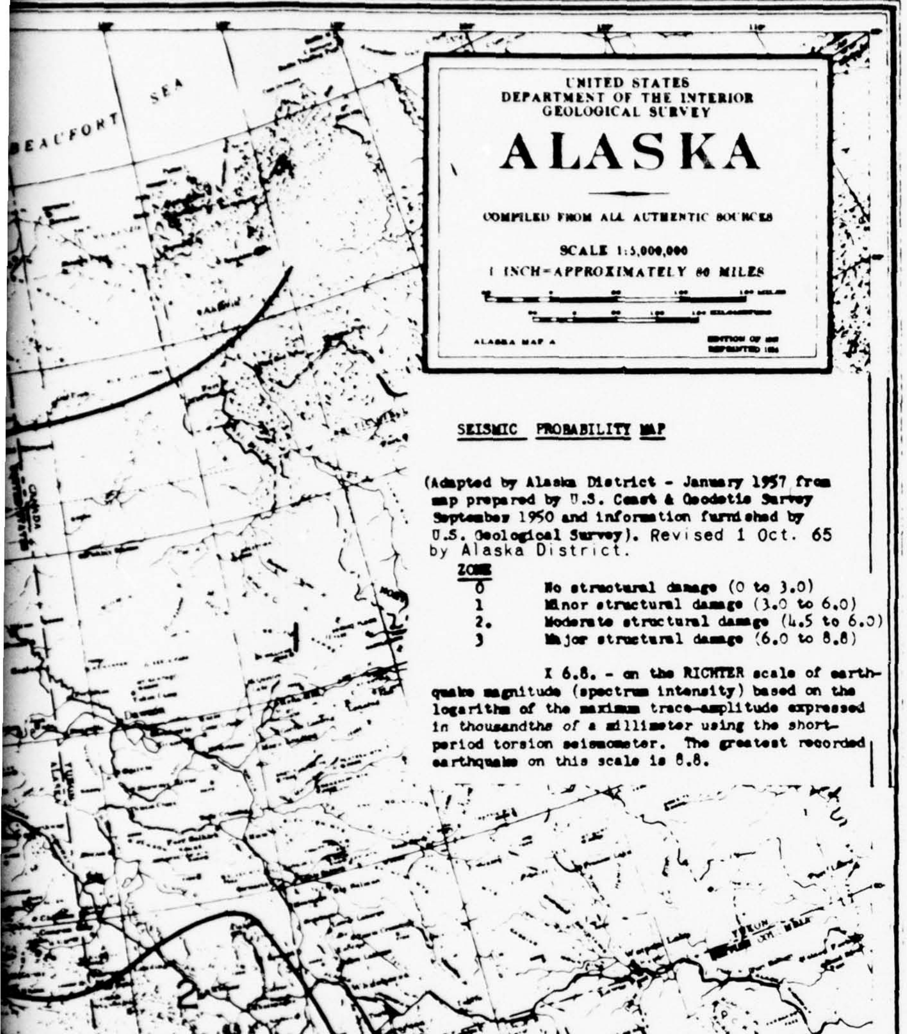
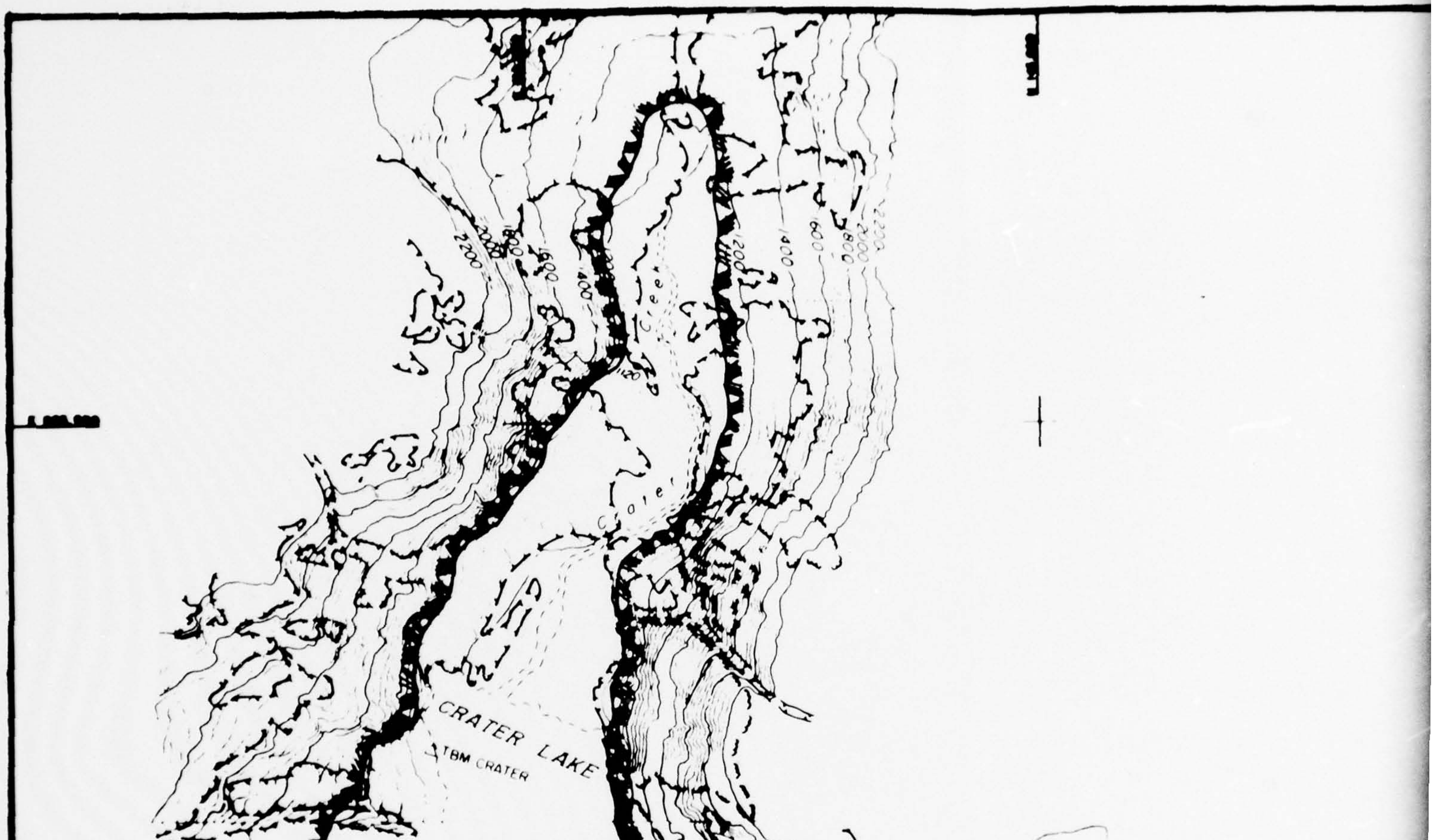
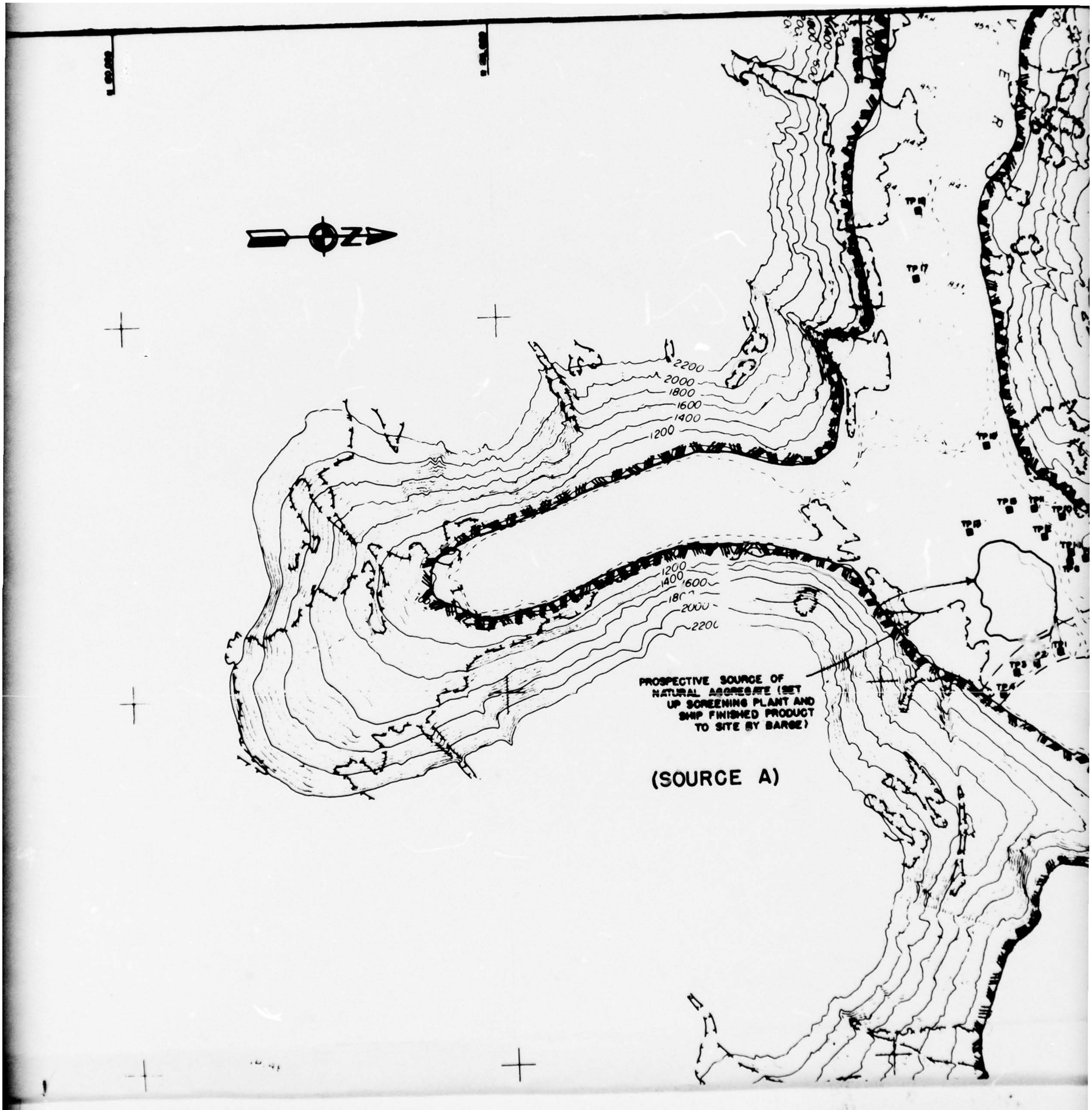


PLATE 5

ALASKA
MAP A

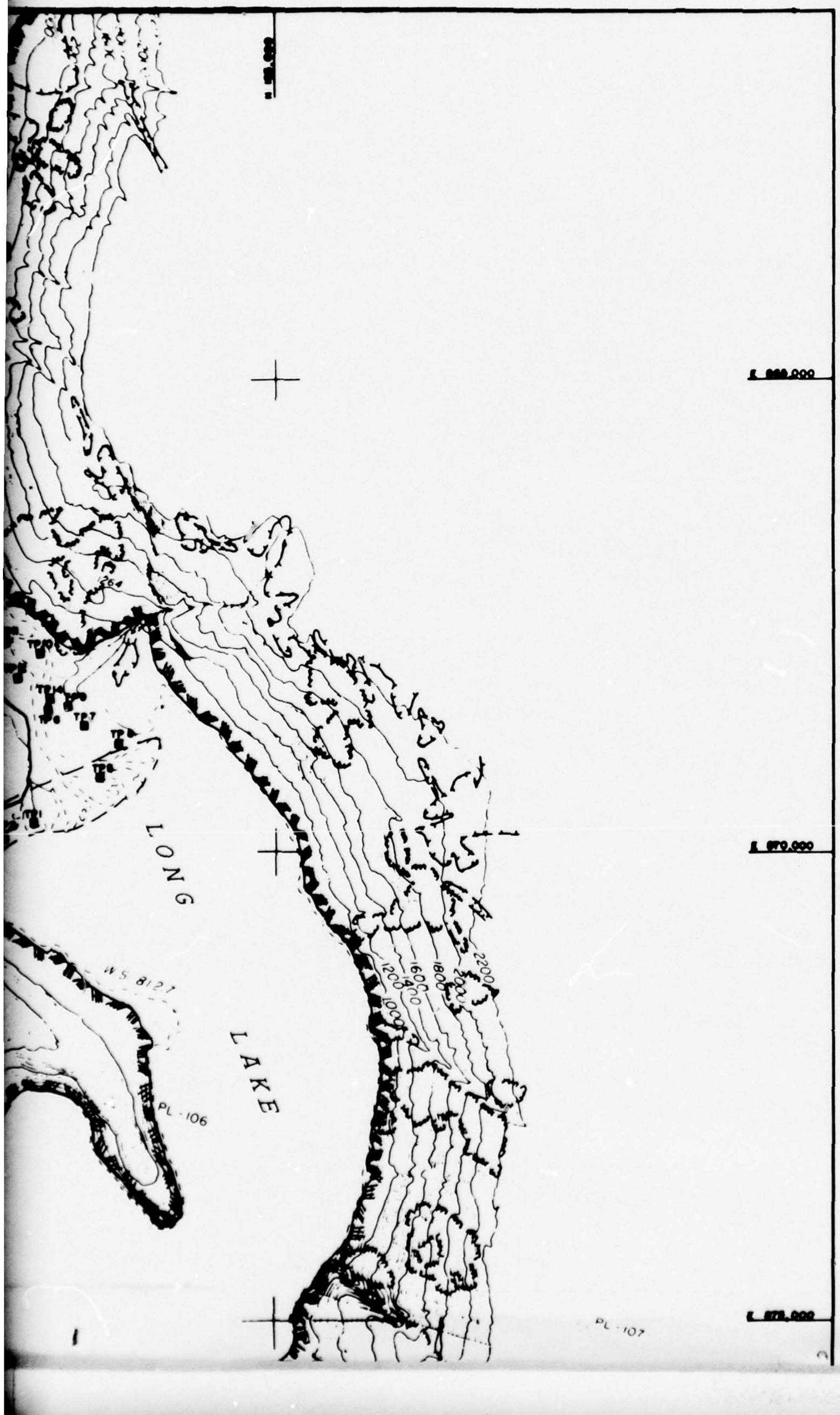
5


CRATER LAKE
TBM CRATER
CRATER

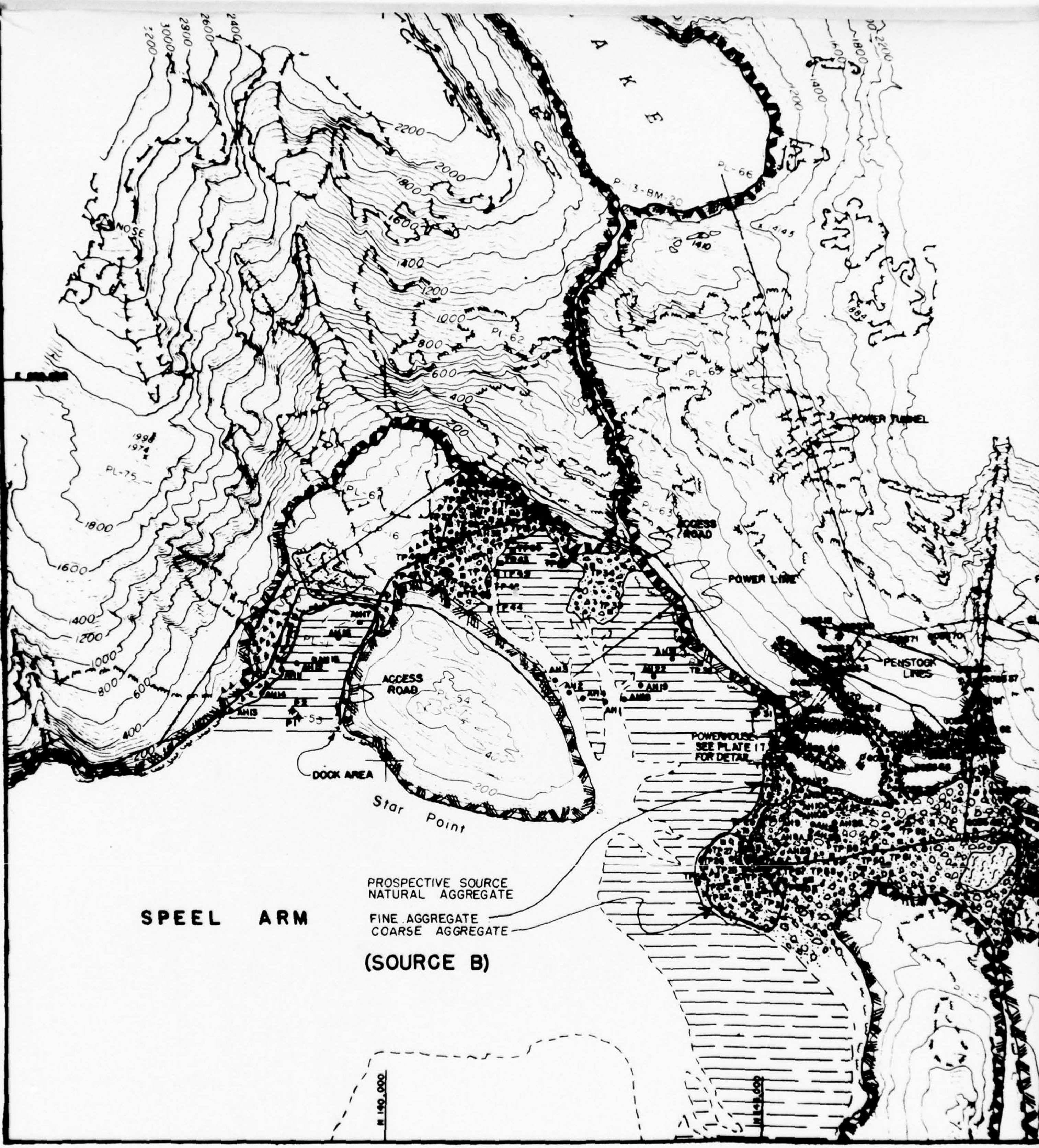


2200
2000
1800
1600
1400
1200
PROSPECTIVE SOURCE OF NATURAL AGGREGATE (SET UP SCREENING PLANT AND SHIP FINISHED PRODUCT TO SITE BY BARGE)
(SOURCE A)

2 3



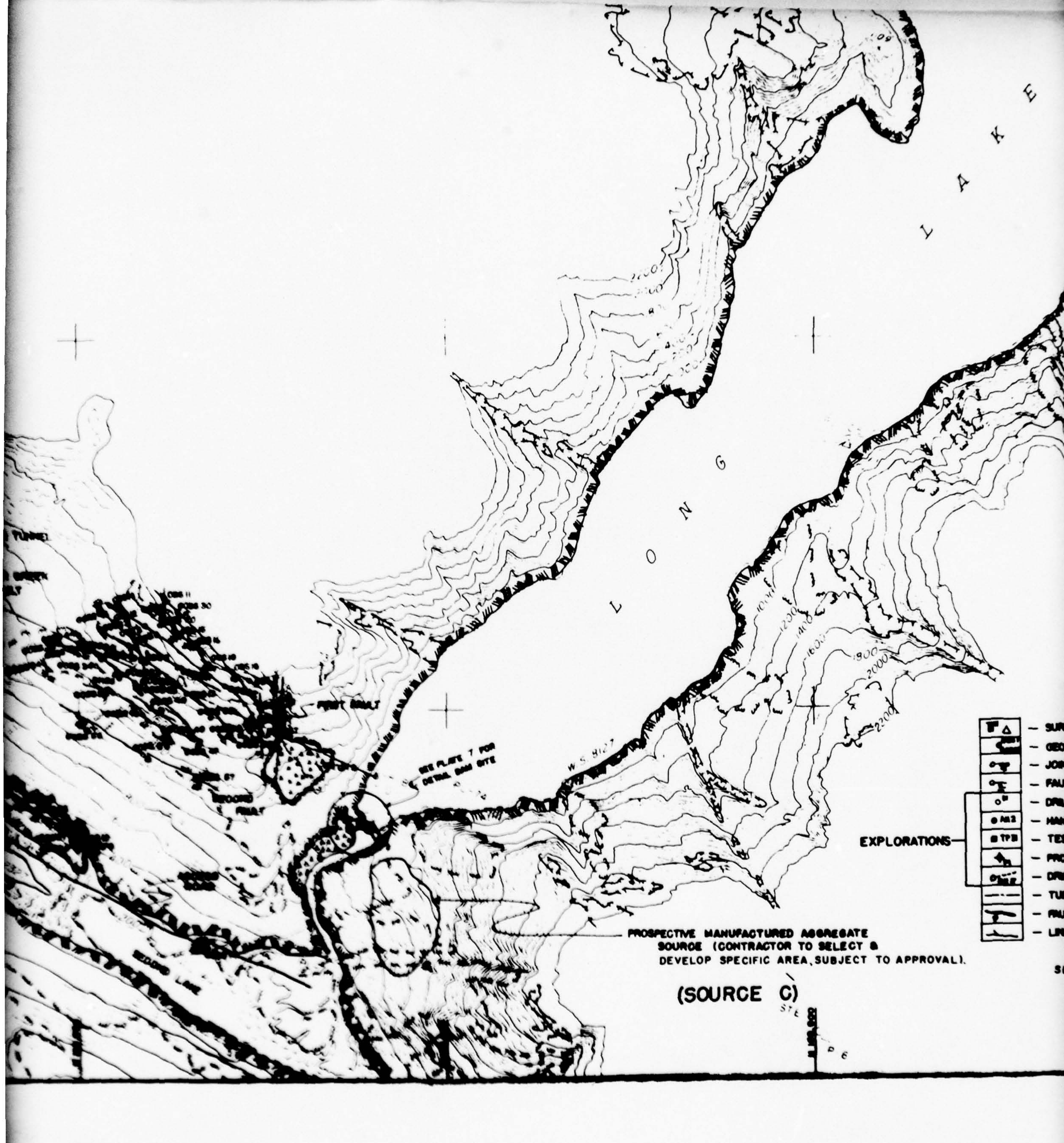
LONG
LAKE
W.S. 8127
PL-106
PL-107
2200
2000
1800
1600
1400
1200
1000
1



PROSPECTIVE SOURCE
NATURAL AGGREGATE
FINE AGGREGATE
COARSE AGGREGATE
(SOURCE B)
SPEEL ARM
Star Point
DOCK AREA
ACCESS ROAD
ACCESS ROAD
POWER LINE
POWER TUNNEL
PENSTOCK LINES
POWERHOUSE SEE PLATE 17 FOR DETAIL
NOSE
P-13-BM-20
PL-66
PL-62
PL-65
PL-63
PL-67
PL-75
PL-54
PL-53

1268-66

4


LONG LAKE
FIRST FAULT
SEE PLATE 7 FOR DETAIL DAM SITE
W.S. 8127
EXPLORATIONS
PROSPECTIVE MANUFACTURED AGGREGATE SOURCE (CONTRACTOR TO SELECT & DEVELOP SPECIFIC AREA, SUBJECT TO APPROVAL).
(SOURCE C)

E 890,000

STEP 2

1000 500 0 1000 2000 3000

E 885,000

TROL POINTS
OBS PT
KE & DIP
NE & DIP
E – (ROCK BIT)
HOLE

E – (WATER JET)
E – (DIAMOND CORE)
E
E
– JOINT – DIKE (BASALT & QTZ FIELDS)
CONTROLLED
TE 6-A FOR REVISED ACCESS ROAD LINE.

— MUD FLATS
— TALUS
— GRAVEL
— ROCK LINE
— MUSKEG
— PEAT & SAND
— VARVED CLAY
— CONTOUR LINE
— OUTLINE OF PROPOSED LONG LAKE DAM
— SNOW AREAS
— WOODED AREAS

N 185,000

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT ALASKA
CORPS OF ENGINEERS
ANCHORAGE ALASKA

DESIGNED Wm B TRAUTWEIN
DRAWN Wm B TRAUTWEIN
CHECKED W.G. Binkley
PREPARED

SNETTISHAM PROJECT, ALASKA
GENERAL DESIGN
GEOLOGIC & EXPLORATION MAP

SUPERVISED N. A. Dixon
CHIEF, GEOLOGY SECTION
SUBMITTED Leland B. Jones
CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED Warren George
CHIEF ENGINEERING DIVISION

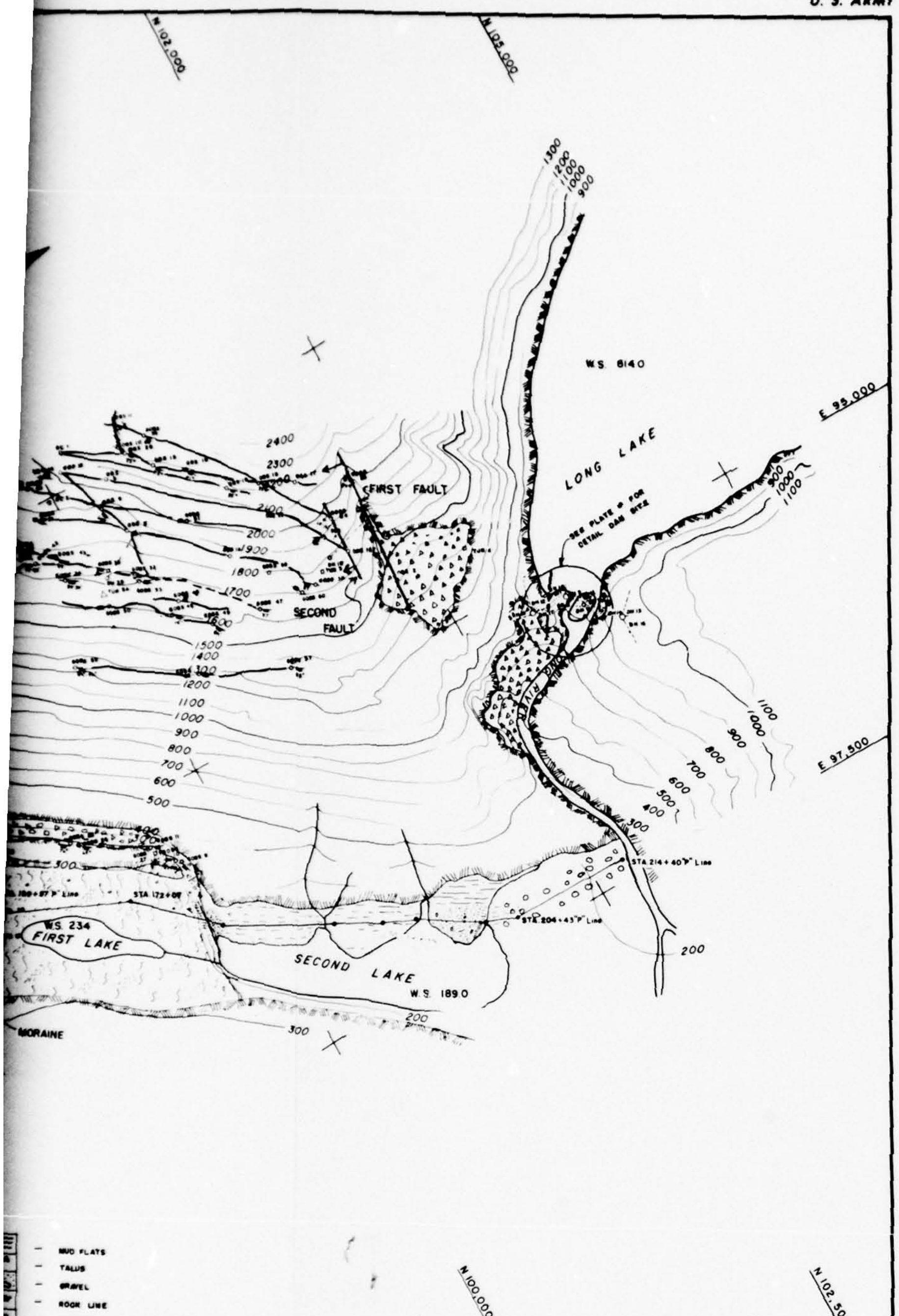
APPROVED Clare Farley
SCALE
DATE 20 Oct 65
FILE NUMBER
SHEET OF

DESIGN MEMORANDUM NO. 7

PLATE 6

CORPS OF ENGINEERS

U. S. ARMY



- MUD FLATS
- TALUS
- GRAVEL
- ROCK LINE
- MUSKEG
- PEAT & SAND
- VARVED CLAY
- CONTOURS LINE
- WATER
- OUTLINE OF PROPOSED LONG LAKE DAM
- SURVEY LINE DAM AXIS (ORIGINAL)
- SURVEY LINE DAM AXIS (SECOND SOUNDING BASE LINE)
- SNOW AREAS
- WOODED AREAS

500 0 500 1000 1500 2000

N 97,500

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

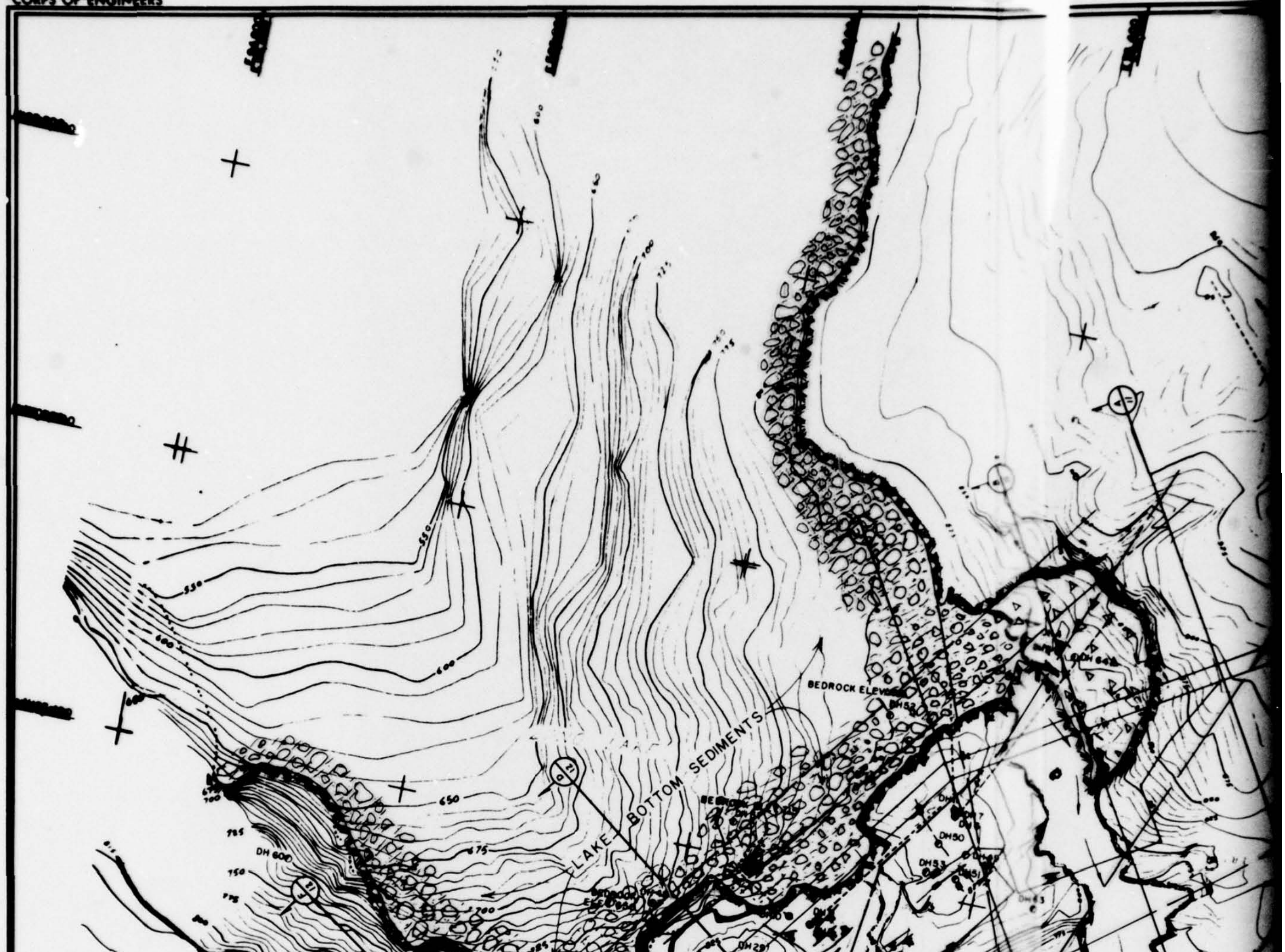
DESIGNED: Wm. Trautwein
DRAWN: W.E.T. RDS
CHECKED: W.G. Binkley
PREPARED:

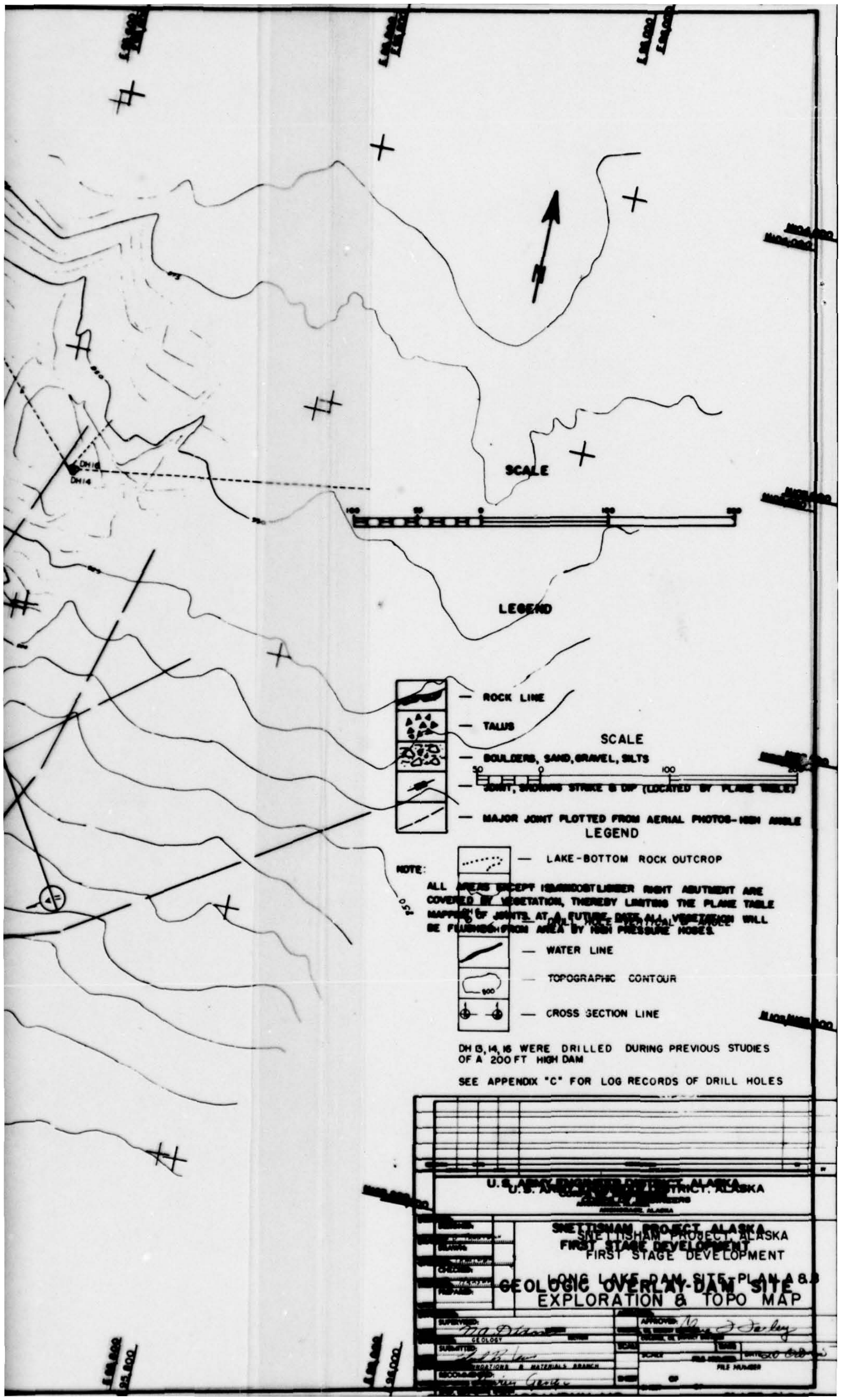
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
GEOLOGIC MAP WITH ACCESS ROAD ALIGNMENT

CHIEF, SUPERVISED: M. G. Dixon — CHIEF, GEOLOGY SECTION
SUBMITTED: — CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED: Harris George — CHIEF, ENGINEERING DIVISION
APPROVED: Clare F. Farley — COLONEL, CE, DISTRICT ENGINEER
SCALE — DATE 30 Oct 63
FILE NUMBER
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 6A

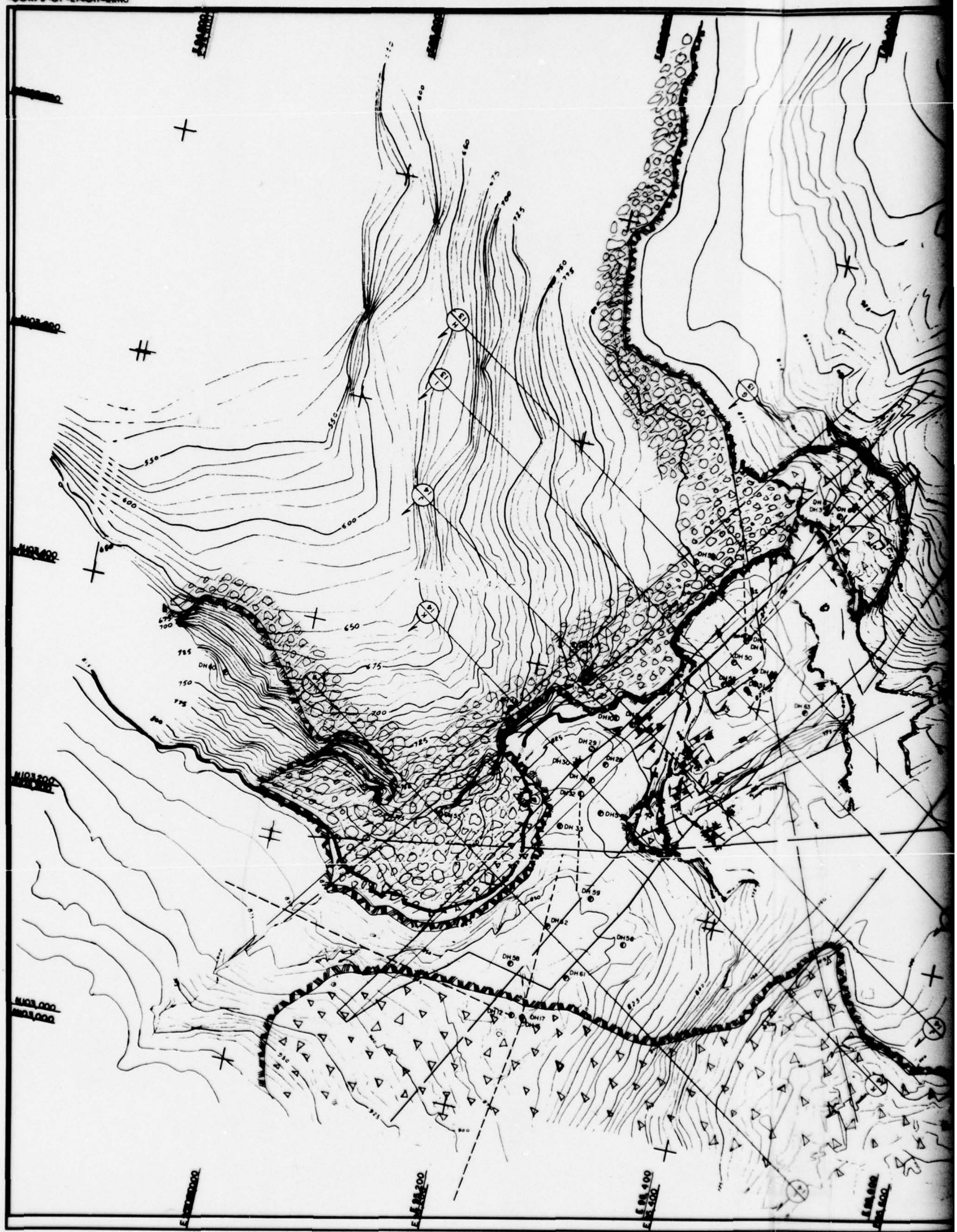
CORPS OF ENGINEERS

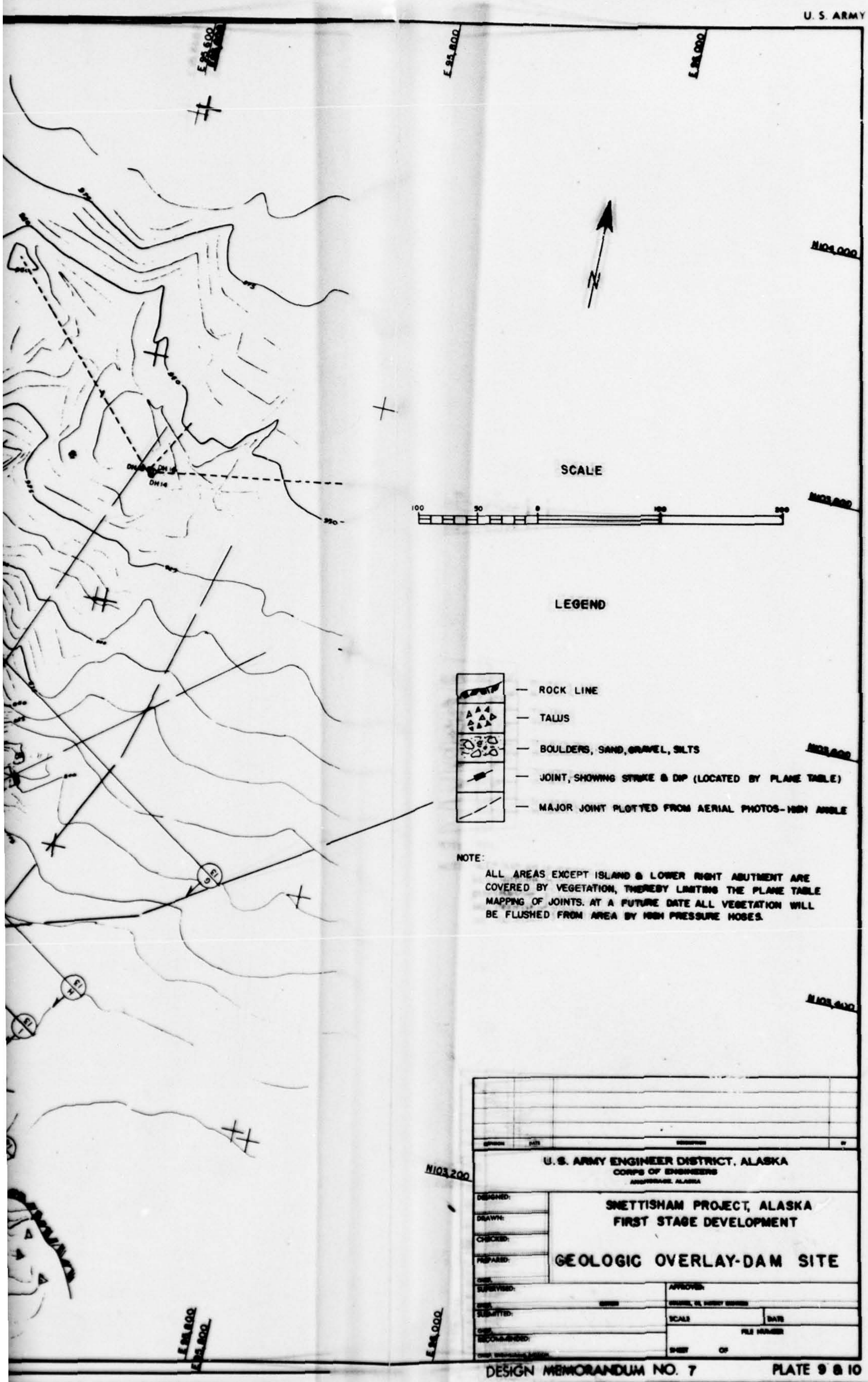




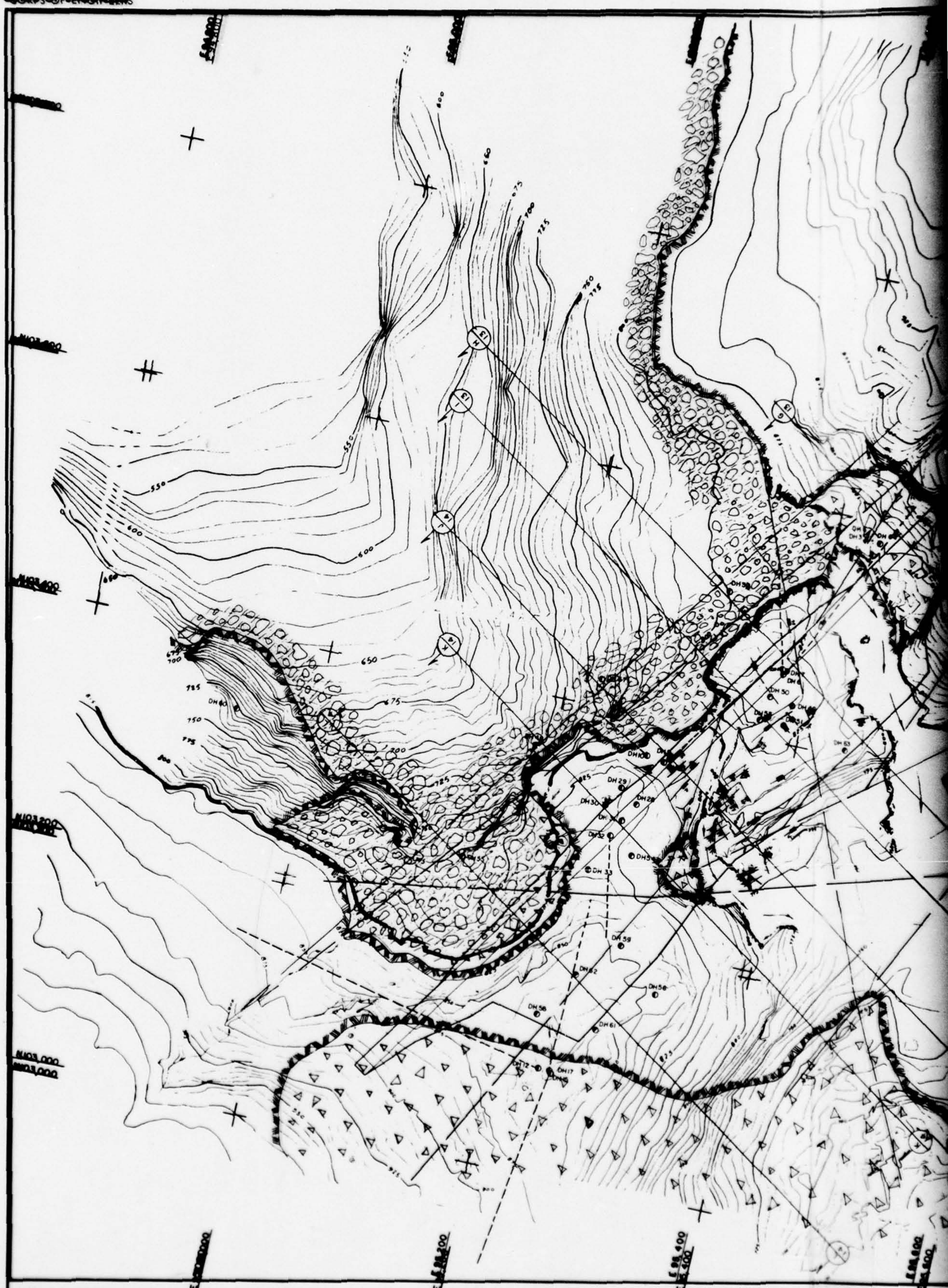
SCALE
LEGEND
ROCK LINE
TALUS
BOULDERS, SAND, GRAVEL, SILTS
MAJOR JOINT PLOTTED FROM AERIAL PHOTOS- HIGH ANGLE
LEGEND
LAKE-BOTTOM ROCK OUTCROP
NOTE:
WATER LINE
TOPOGRAPHIC CONTOUR
CROSS SECTION LINE
DH 13,14,16 WERE DRILLED DURING PREVIOUS STUDIES OF A 200 FT HIGH DAM
SEE APPENDIX "C" FOR LOG RECORDS OF DRILL HOLES
U.S. ARMY ENGINEER DISTRICT, ALASKA
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
GEOLOGIC OVERLAY-DAM SITE
EXPLORATION & TOPO MAP

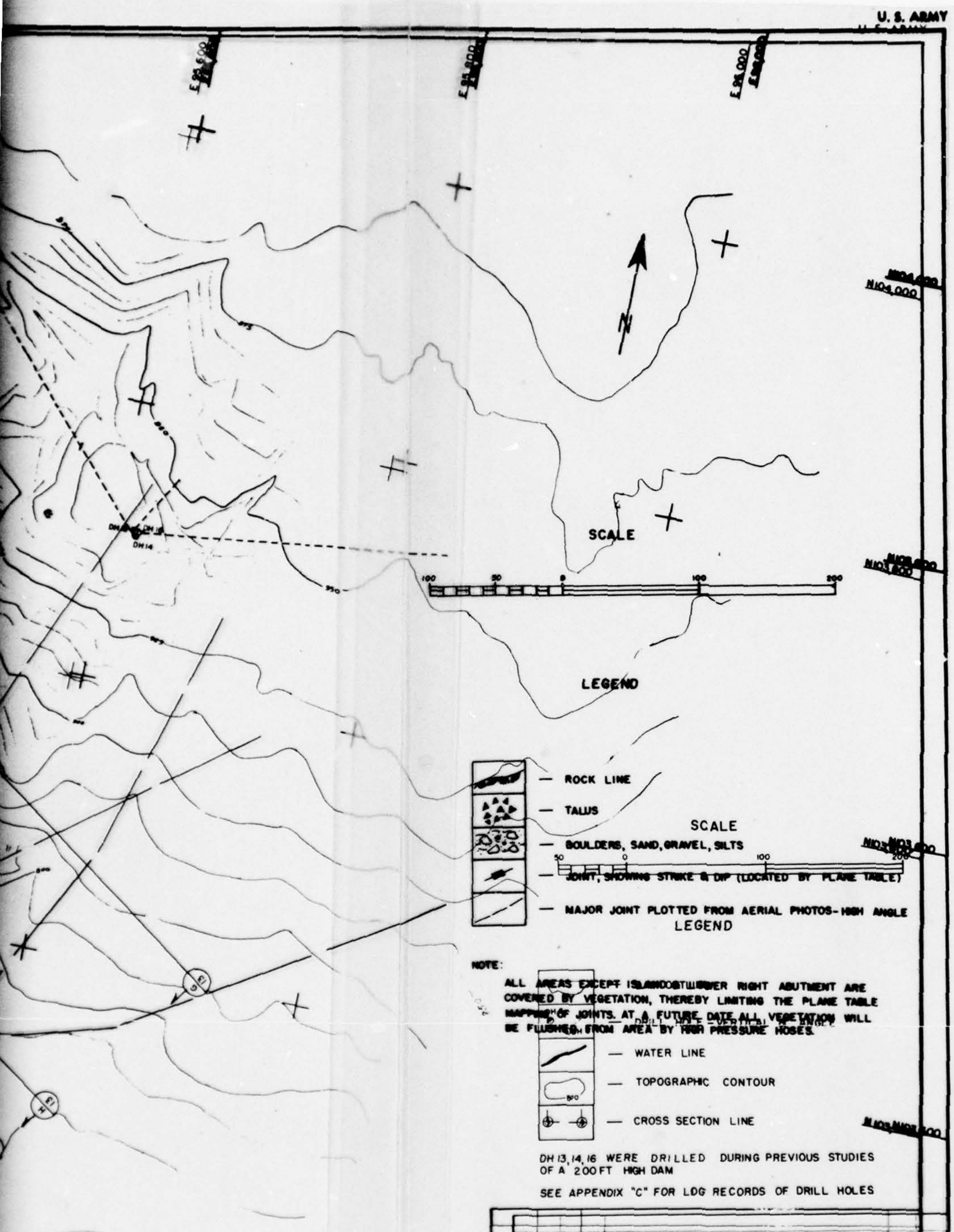
CORPS OF ENGINEERS




N104,000
N103,800
N103,600
N103,400
N103,200
SCALE
100
50
0
100
200
LEGEND
— ROCK LINE
— TALUS
— BOULDERS, SAND, GRAVEL, SILTS
— JOINT, SHOWING STRIKE & DIP (LOCATED BY PLANE TABLE)
— MAJOR JOINT PLOTTED FROM AERIAL PHOTOS- HIGH ANGLE
NOTE:
ALL AREAS EXCEPT ISLAND & LOWER RIGHT ABUTMENT ARE COVERED BY VEGETATION, THEREBY LIMITING THE PLANE TABLE MAPPING OF JOINTS. AT A FUTURE DATE ALL VEGETATION WILL BE FLUSHED FROM AREA BY HIGH PRESSURE HOSES.
DH14
U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA
DESIGNED:
DRAWN:
CHECKED:
PREPARED:
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
GEOLOGIC OVERLAY-DAM SITE
SUPERVISED:
APPROVED:
SUBMITTED:
SCALE
DATE
RECOMMENDED:
FILE NUMBER
SHEET
OF

CORPS OF ENGINEERS

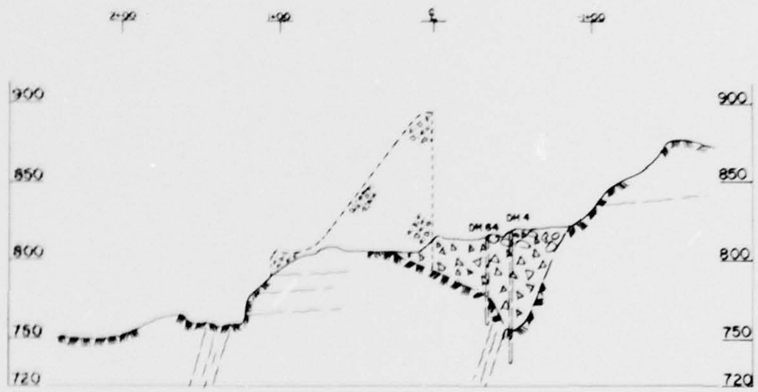




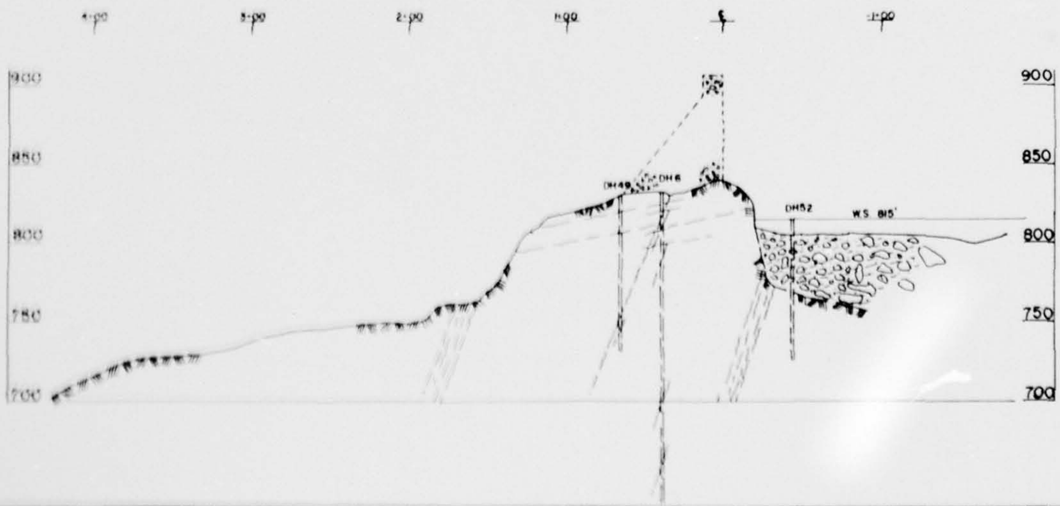
U. S. ARMY
SCALE
LEGEND
ROCK LINE
TALUS
BOULDERS, SAND, GRAVEL, SILTS
JOINT, SHOWING STRIKE & DIP (LOCATED BY PLANE TABLE)
MAJOR JOINT PLOTTED FROM AERIAL PHOTOS- HIGH ANGLE
NOTE:
ALL AREAS EXCEPT ISLAND AND RIGHT ABUTMENT ARE COVERED BY VEGETATION, THEREBY LIMITING THE PLANE TABLE MAPPING OF JOINTS. AT A FUTURE DATE ALL VEGETATION WILL BE FLUSHED FROM AREA BY HIGH PRESSURE HOSES.
DRILL HOLE VERTICAL
WATER LINE
TOPOGRAPHIC CONTOUR
CROSS SECTION LINE
DH 13, 14, 16 WERE DRILLED DURING PREVIOUS STUDIES OF A 200 FT HIGH DAM
SEE APPENDIX "C" FOR LOG RECORDS OF DRILL HOLES
U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA
SNETTISHAM PROJECT ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM SITE - PLAN C
GEOLOGIC OVERLAY-DAM SITE
EXPLORATION & TOPO MAP
DESIGNED
DRAWN
CHECKED
PREPARED
SUPERVISED
SUBMITTED
RECOMMENDED
APPROVED
SCALE
FILE NUMBER
DESIGN MEMORANDUM NO. 7
PLATE 8

SECTION A

SECTION B



SECTION C



U. S. ARMY

SCALE

50 0 100 200

LEGEND

— ROCK LINE

— TALUS

— GRAVEL, BOULDERS, SAND, SILT

— CONCRETE

— DRILL HOLE, PROJECTED (DIAMOND CORE)

— SIGNIFICANT JOINTS

— LAKE LEVEL

FOUNDATION DESIGN PLANS "A" & "B"

REFERENCES: Location of sections, PLATE 7
Grouting plan, PLATE 15
Logs of borings, APPENDIX C

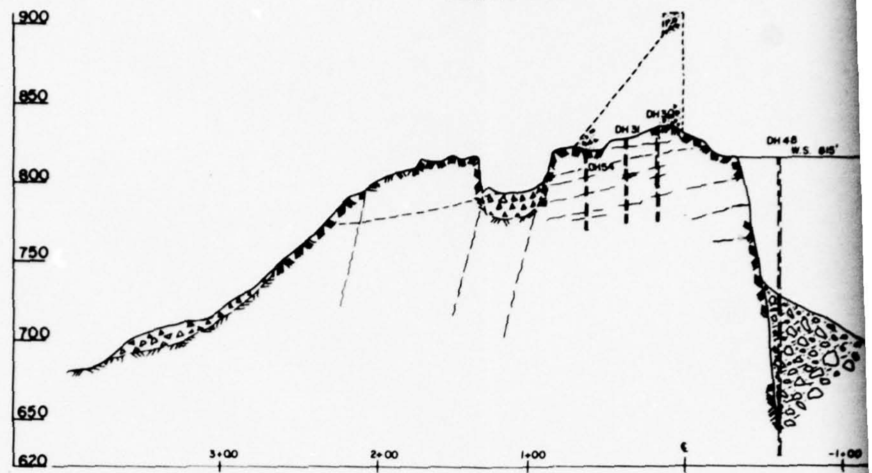
REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE ALASKA

DESIGNED: Wm. Trautwein	SNETTISHAM PROJECT, ALASKA
DRAWN: W G Binkley	FIRST STAGE DEVELOPMENT
CHECKED: D.V. Lachner	LONG LAKE DAM SITE
PREPARED: CHIEF	GEOLOGIC SECTIONS
SUPERVISED: V.A. Dixon CHIEF, GEOLOGY SECTION	APPROVED: Clare F. Farley COLONEL, CE, DISTRICT ENGINEER
SUBMITTED: [illegible] CHIEF, FOUNDATIONS & MATERIALS BRANCH	SCALE / DATE 20 Oct 65
RECOMMENDED: Warren George CHIEF, ENGINEERING DIVISION	FILE NUMBER / SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 11

SECTION D



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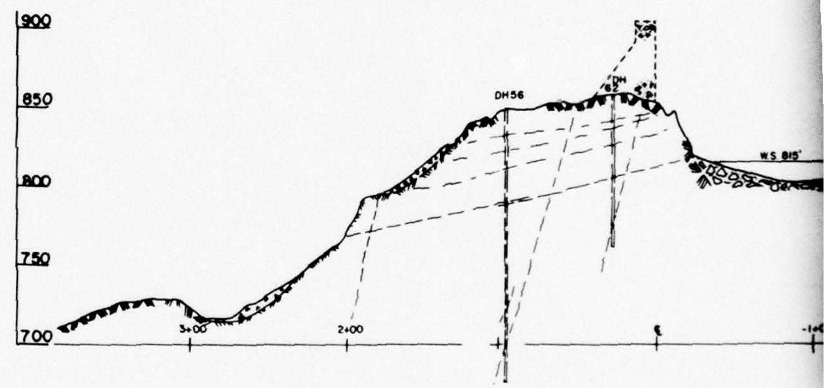

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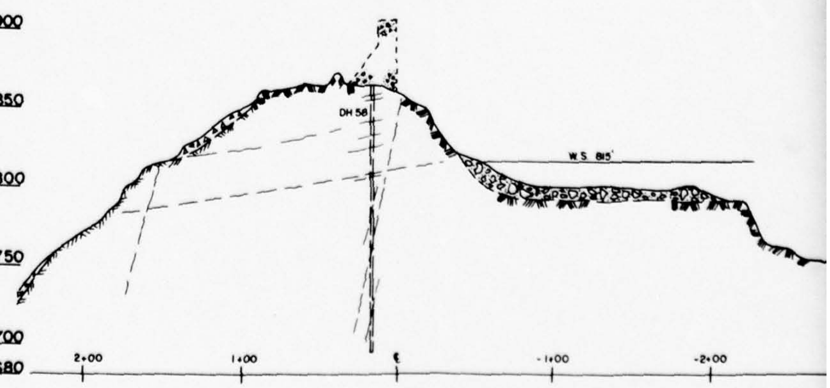
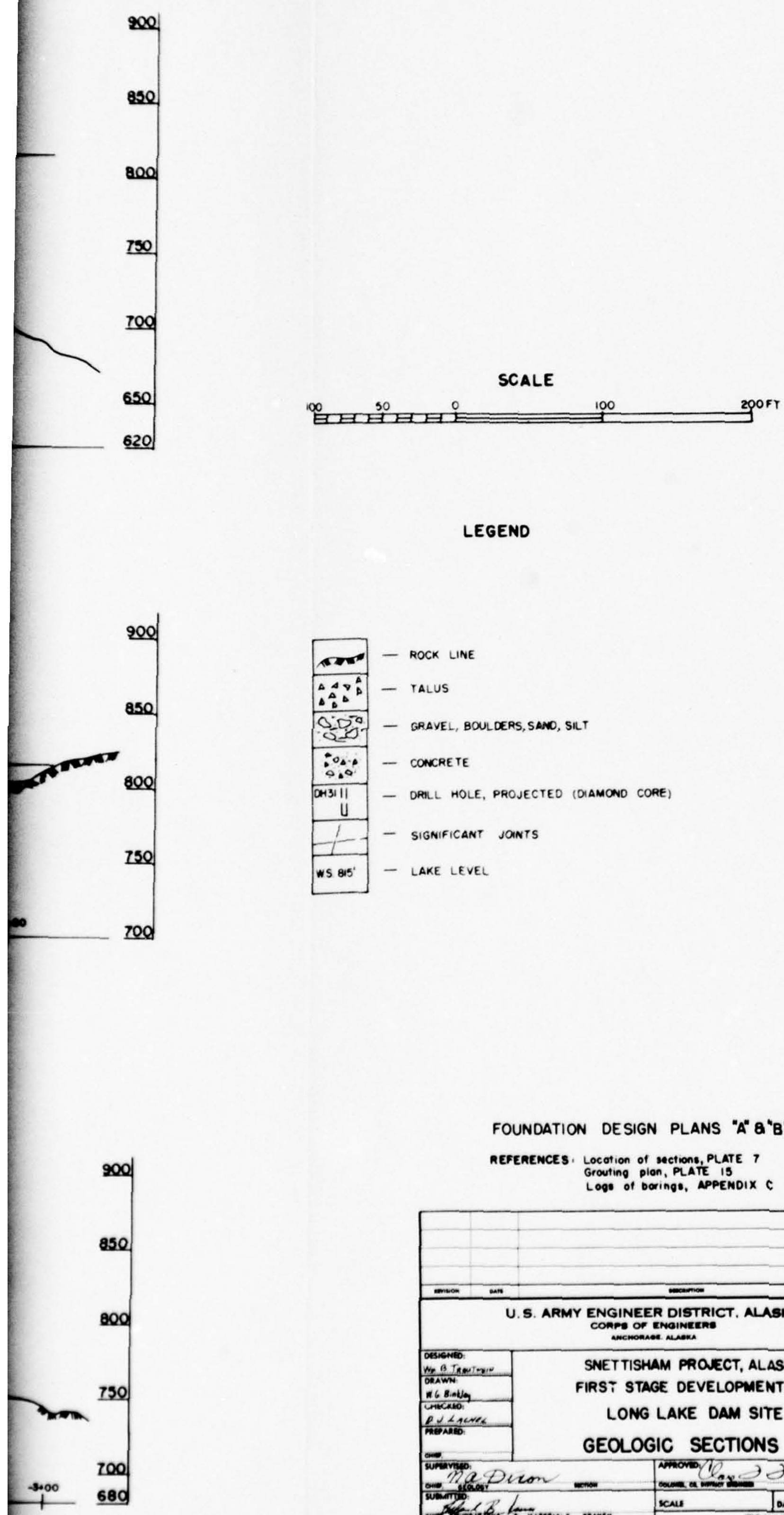


U. S. ARMY



FOUNDATION DESIGN PLANS "A" & "B"

REFERENCES: Location of sections, PLATE 7
Grouting plan, PLATE 15
Logs of borings, APPENDIX C

REVISION | DATE | DESCRIPTION | BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

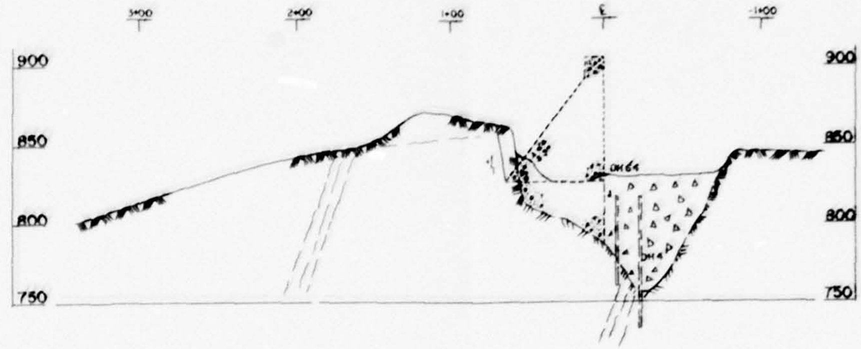
DESIGNED: Wm B Troutwin
DRAWN: W G Binkley
CHECKED: D J Lachel
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM SITE
GEOLOGIC SECTIONS

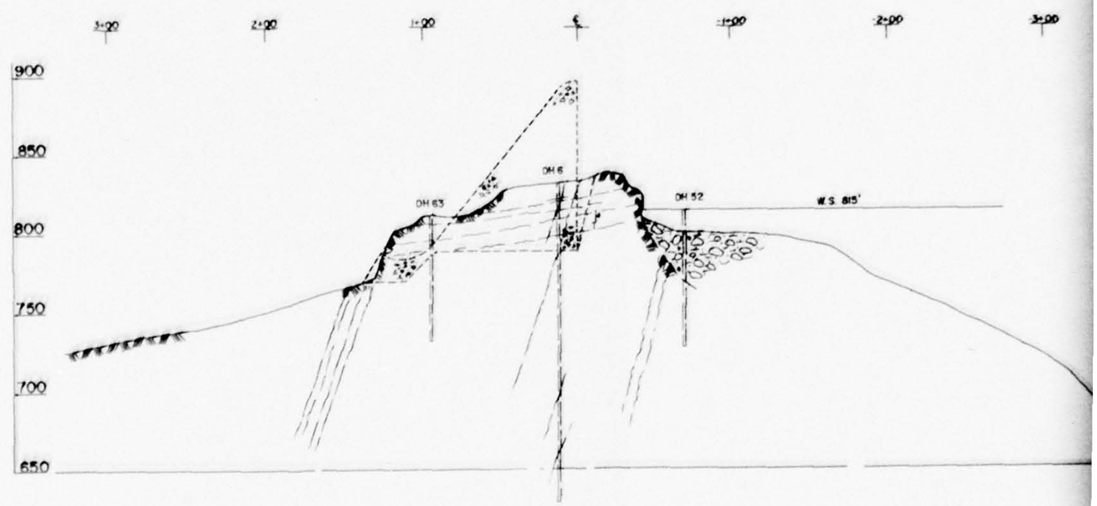
SUPERVISED: M A Dixon, CHIEF, GEOLOGY SECTION
SUBMITTED: CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED: CHIEF, ENGINEERING DIVISION
APPROVED: COLONEL, CE, DISTRICT ENGINEER
SCALE
DATE 20 Oct 65
FILE NUMBER
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 12

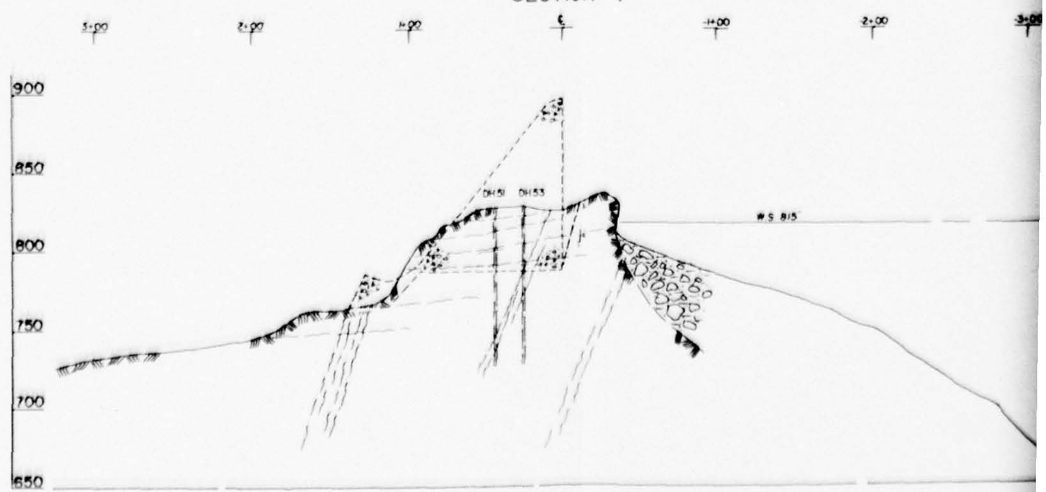
SECTION G



SECTION H



SECTION I

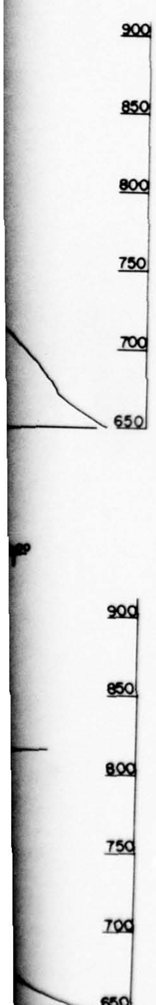


U. S. ARMY

SCALE

100 50 0 100 200

LEGEND



— ROCK LINE

— TALUS

— GRAVEL, BOULDERS, SAND, SILT

— CONCRETE

DH 52 — DRILL HOLE, PROJECTED (DIAMOND CORE)

— SIGNIFICANT JOINTS

W.S. 815' — LAKE LEVEL

FOUNDATION DESIGN PLAN "C"

REFERENCES:
Location of sections, PLATE 8
Grouting plan, PLATE 16
Logs of borings, APPENDIX C

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

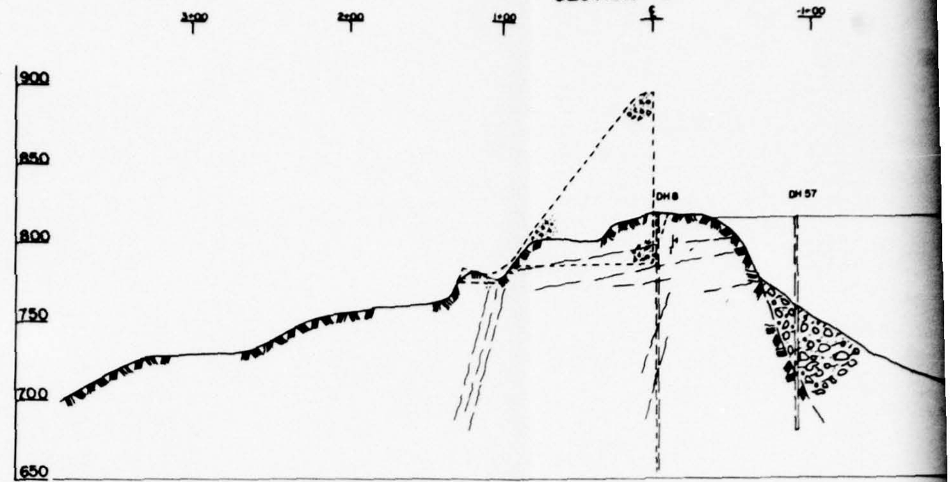
DESIGNED: Wm. R. Trenthoir
DRAWN:
CHECKED: D. L. Lachel
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM SITE
GEOLOGIC SECTIONS

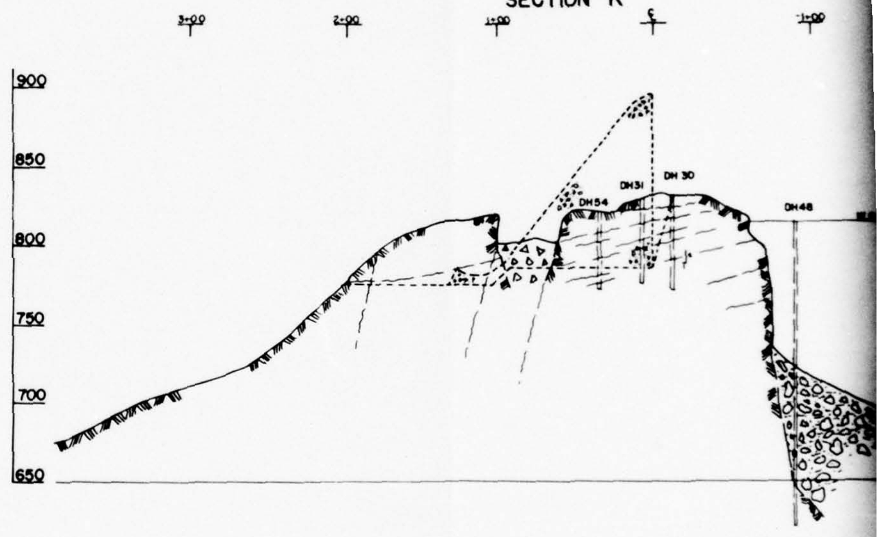
SUPERVISED: N. A. Dixon — CHIEF, GEOLOGY SECTION
SUBMITTED: CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED: CHIEF, ENGINEERING DIVISION
APPROVED: COLONEL, CE, DISTRICT ENGINEER
SCALE
DATE 30 Oct 65
FILE NUMBER
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 13

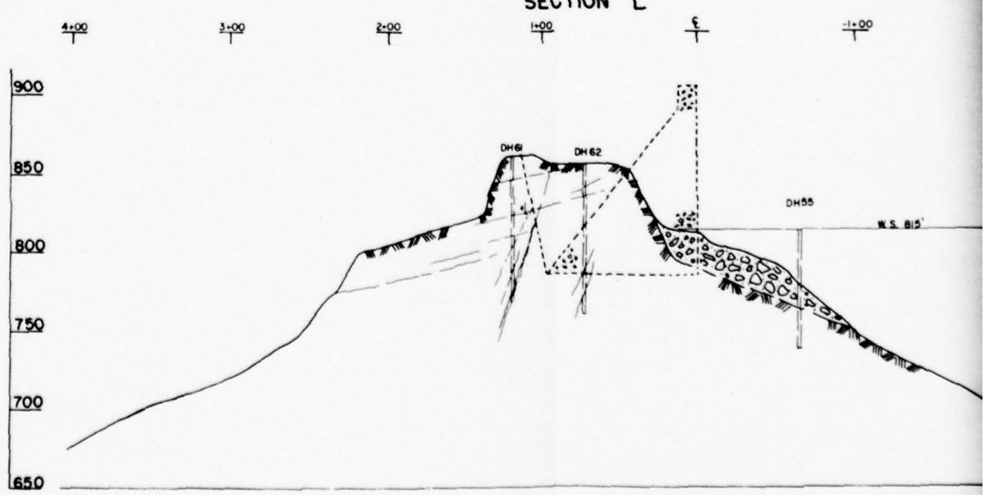
SECTION J



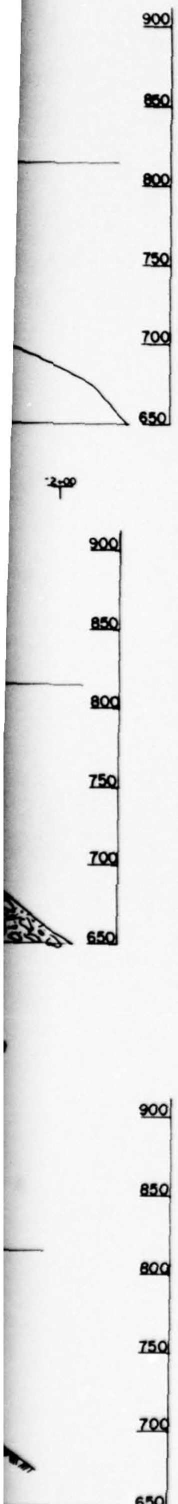
SECTION K



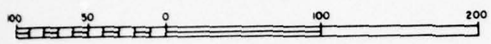
SECTION L



U. S. ARMY



SCALE



LEGEND

- ROCK LINE
- TALUS
- GRAVEL, BOULDERS, SAND, SILT
- CONCRETE
- DH 52 — DRILL HOLE, PROJECTED (DIAMOND CORE)
- SIGNIFICANT JOINTS
- W.S. 815 — LAKE LEVEL

FOUNDATION DESIGN PLAN "C"

REFERENCES: Location of sections, PLATE 6
Grouting plan, PLATE 16
Logs of borings, APPENDIX C

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: Wm. Trautwein
DRAWN: W.E. Binkley
CHECKED: D. Hansen
PREPARED:

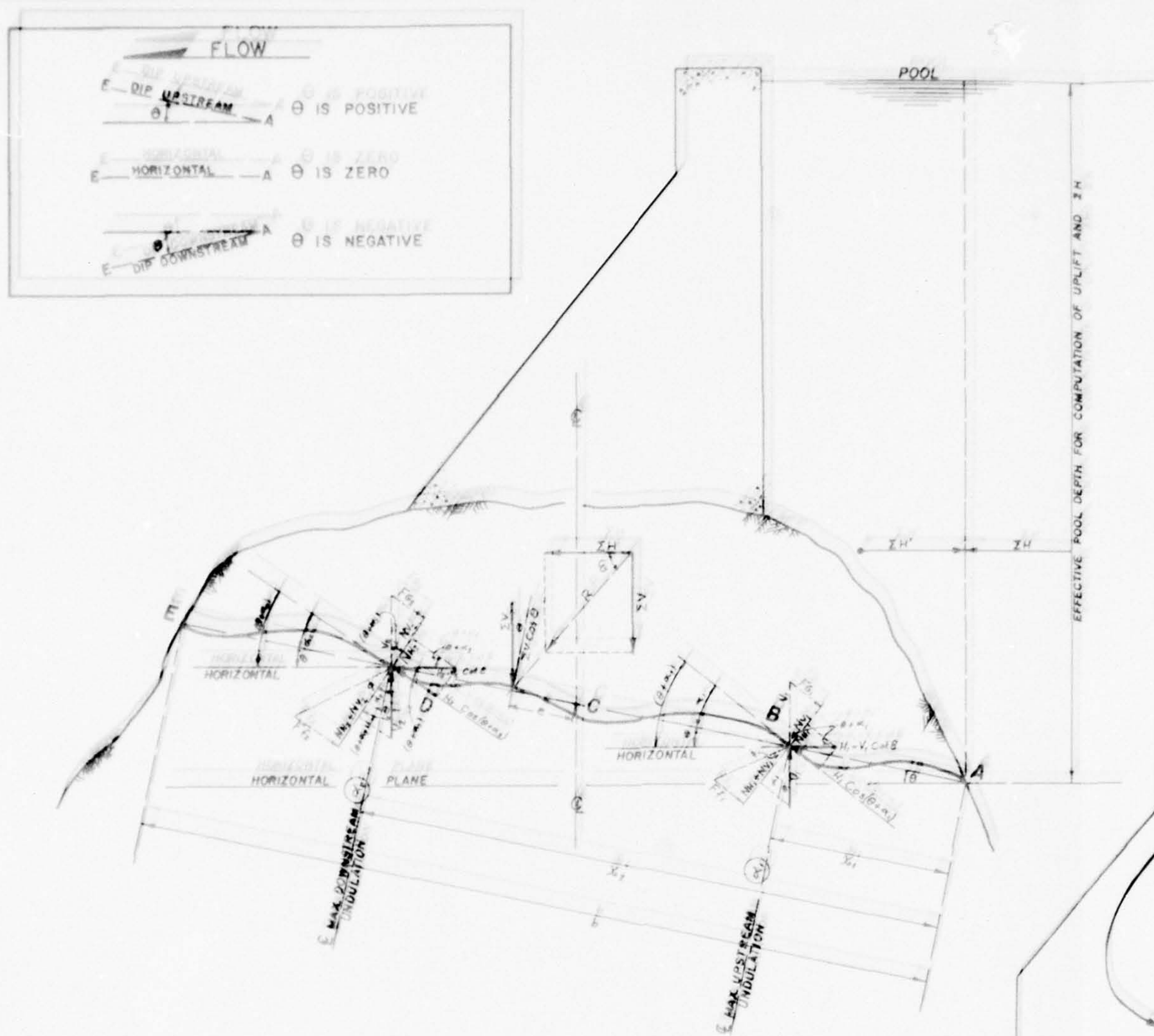
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM SITE
GEOLOGIC SECTIONS

SUPERVISED: N.A. Dixon — CHIEF, GEOLOGY SECTION
APPROVED: — COLONEL, CE, DISTRICT ENGINEER
SUBMITTED: — CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED: — CHIEF, ENGINEERING DIVISION
SCALE
DATE 20 Oct 63
FILE NUMBER
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 14

2

CORPS OF ENGINEERS



DEFINITIONS & EXPLANATORY NOTES

(1) THE ABOVE CONCRETE GRAVITY DAM IS CONSTRUCTED UPON A RELATIVELY NARROW RIDGE OF HARD IGNEOUS ROCK. THE DAM IS POSITIONED SO THAT THE RESULTANT "R" FALLS WITHIN THE MIDDLE THIRD OF PLANE AE. RESULTANT "R" MAKES AN ANGLE OF "β" WITH THE HORIZONTAL AND INTERSECTS PLANE AE AT "e" DISTANCE DOWNSTREAM FROM CENTROIDAL POINT "C" OF PLANE AE.

(2) JOINT SURFACE ABCDE IS AN UNDULATING LOW ANGLE JOINT WHICH "DAYLIGHTS" OUT ON SLOPING ROCK SURFACES IN BOTH UPSTREAM AND DOWNSTREAM DIRECTIONS. AVERAGE PLANE AE OF THIS JOINT SURFACE MAKES AN ANGLE "θ" WITH A HORIZONTAL PLANE. ANGLE θ IS POSITIVE FOR UPSTREAM DIP (AS SHOWN), ZERO FOR THE CASE OF AN HORIZONTAL SURFACE, AND NEGATIVE FOR THE CASE OF DOWNSTREAM DIP OF PLANE AE. (NET ANGLE (θ + α) COULD THEREFORE BE NEGATIVE IN CASES OF STEEP DOWNSTREAM DIP TO PLANE AE).

(3) UNDULATION SLOPES α_1, AND α_2 (MEASURED FROM PLANE AE) ARE THE MAXIMUM UNDULATION SLOPES WHICH EXIST ON JOINT SURFACE ABCDE. ANGLE α_1 IS THE MAXIMUM UNDULATION SLOPE UPSTREAM FROM CENTROIDAL POINT C, AND ANGLE α_2 IS THE MAXIMUM UNDULATION SLOPE DOWNSTREAM FROM CENTROIDAL POINT C. ANY DOWNSTREAM MOVEMENT OF ROCK MASS AND DAM ABOVE JOINT ABCDE MUST: (a) RIDE UP AND OVER SLOPES α_1 AND α_2 (SEE MODEL STUDY); OR (b) SHEAR THROUGH ALL ROCK UNDULATIONS ALONG PLANE AE. CONSIDERATION OF ENERGY CONSIDERATIONS WOULD USUALLY PROHIBIT THE ALTERNATE OF SHEAR FAILURE THROUGH HARD IGNEOUS ROCK (5000 psi +) EXCEPT IN CASES WHERE ANGLE (θ + α) → 90° FOR WHICH THE TANGENT FUNCTION → ∞. METHOD OF FAILURE IS THEREFORE A FUNCTION OF INHERENT SHEAR STRENGTH AND (θ + α).

(4) TAN φ IS THE COEFFICIENT OF STATIC FRICTION ALONG JOINT SURFACE ABCDE, BUT AT α_1 AND α_2 SLOPES SPECIFICALLY.

(5) DISTANCES X_1, AND X_2 ARE THE DISTANCES FROM A TO THE CENTERS OF UNDULATION SLOPES α_1 AND α_2, RESPECTIVELY. FORCES V_1 AND V_2 ARE THE COMPUTED REACTIONS AT α_1 AND α_2 RESPECTIVELY WHICH TOGETHER SUPPORT [illegible] LOADING PROFILE. ΣV IS THE ALGEBRAIC SUMMATION OF ALL VERTICAL FORCES ON JOINT SURFACE ABCDE INCLUDING UPLIFT, AND ACTS AT "e" DISTANCE FROM CENTROIDAL POINT C. A CORRECTION WOULD BE REQUIRED FOR LARGE VALUES OF (θ + α) BECAUSE OF THE DEVIATION OF THE UPLIFT FORCE ALONG JOINT ABCDE FROM VERTICAL.

(6) WITH MOVEMENT DEVELOPING ALONG JOINT SURFACE ABCDE, LOADING CONDITIONS ALONG THE JOINT BEGIN TO CHANGE FROM COMPRESSION LOADING AT ALL POINTS ALONG THE JOINT SURFACE (PROVIDED RESULTANT FALLS IN MIDDLE THIRD), TO CONCENTRATED LOADING AT THE CENTER "POINTS" OF THE TWO MAXIMUM UNDULATION SLOPES (α_1 AND α_2) ON EITHER SIDE OF CENTROIDAL POINT C. (SEE MODEL TEST PHOTO). SINCE THE FRICTIONAL FORCE IS THEORETICALLY INDEPENDENT OF AREA, IT WOULD MAKE LITTLE DIFFERENCE IF THE ENTIRE LOAD (ΣV) WERE UNIFORMLY DISTRIBUTED ALONG THE ENTIRE JOINT AREA, OR SUPPORTED AT TWO POINTS (OR LIMITED AREAS) ALONG THE JOINT SURFACE. BY COMPUTING LOAD REACTIONS AT THESE TWO ESTABLISHED POINTS OF SUPPORT, IT IS POSSIBLE TO DETERMINE THE FOUR RESISTANCE FORCES (2 FROM FRICTION AND 2 FROM GRAVITY) WHICH CAN BE MOBILIZED TO RESIST MOVEMENT UP SLOPES (θ + α_1) AND (θ + α_2) FROM THE DRIVING FORCE ΣH (HYDRAULIC, EARTHQUAKE, ICE, AND WIND).

(7) SINCE THE BORE HOLE CAMERA MEASURES JOINT ANGLES FROM AN HORIZONTAL PLANE, THE COMPOSITE ANGLE (θ + α) CAN BE MEASURED DIRECTLY BY THE CAMERA AS IN THE SKETCH ABOVE. (ANGLE θ, OF THE OVERALL JOINT PLANE IS DETERMINED MORE ACCURATELY FROM GEOLOGIC MAPPING.)

(8) INASMUCH AS IT IS NOT USUALLY PRACTICAL TO COMPLETELY DEFINE ANY JOINT SURFACE IN SUFFICIENTLY PRECISE DETAIL TO APPLY THE COMPLETE GENERAL FORMULA DERIVED ON THIS PAGE, THE ABBREVIATED ONE STATED AT THE BOTTOM OF THE PAGE WOULD APPEAR TO BE MORE APPLICABLE. THIS LATTER FORMULA IS BASED ON THE SIMPLIFIED ASSUMPTION THAT ALL UNDULATIONS ALONG THE JOINT SURFACE ARE EQUAL TO ONE AVERAGE UNDULATION ANGLE AS DETERMINED FROM A STEREO NET PLOT OF MANY LOW ANGLED JOINTS, MEASURED FROM A BORE HOLE PHOTO SURVEY OF THE AREA. THIS PRACTICE SHOULD BE CONSERVATIVE SINCE IT HAS BEEN DEMONSTRATED IN MODEL STUDIES (SEE PHOTOS 31 THRU 33) THAT MAXIMUM AND NOT AVERAGE ANGLES ARE BROUGHT INTO PLAY IN RESISTING MOVEMENT ALONG THE LOW ANGLED JOINTS.

V_1 and V_2 are reaction[illegible]

Taking Moments About [illegible]

$V_1 \cos\theta X_1 + V_2 \cos\theta X_2 = $ [illegible]

$V_1 + V_2 = \Sigma V$ (Vert[illegible]

$V_2 = \Sigma V - V_1$

$V_1 X_1 + (\Sigma V - V_1) X_2 = \Sigma V$ [illegible]

$V_1 X_1 + \Sigma V X_2 - V_1 X_2 = \Sigma V$ [illegible]

$V_1 (X_1 - X_2) = \Sigma V(-X_2 +$ [illegible]

$V_1 = \Sigma V \left[\frac{-X_2 + \frac{b}{2} + \ldots}{X_1 - X_2} \right.$ [illegible]

$N_{V_1} = V_1 \cos(\theta + \alpha_1)$

$N_{H_1} = V_1 \sin(\theta + \alpha_1) \cot\theta$ [illegible]

$\therefore N_{V_1} + N_{H_1} = V_1 [\cos$ [illegible]

$\tan\phi = F_{f_1} / N_{V_1} + N_{H_1}$ [illegible]

$\therefore F_{f_1} = V_1 [\cos(\theta + \alpha_1)$ [illegible]

Similarly $F_{f_2} = V_2 [\cos(\theta + \alpha_2)$ [illegible]

$F_{G_1} = V_1 \sin(\theta + \alpha_1)$ [illegible]

Similarly $F_{G_2} = V_2 \sin(\theta + \alpha_2)$ [illegible]

Coef[illegible]

Coefficient [illegible]

$\therefore \Sigma H = \Sigma V \left[\frac{-X_2 + \frac{b}{2} + e}{X_1 - X_2} \right.$ [illegible]

SAFETY FACTOR $S.F. = \left[\Sigma V \right.$ [illegible]

If Average Angles [illegible] Measured By Bore [illegible]

$S.F. = \left[\Sigma V \left[(\right. \right.$ [illegible]

U. S. ARMY

... from load ΣV at the 2 maximum undulation points α_1 and α_2 respectively

..."A"

$= \Sigma V \cos\theta(\frac{b}{2}+e)$

...rtical Equilibrium)

$\Sigma V(\frac{b}{2}+e)$

$\frac{\Sigma V b}{2} + \Sigma Ve$

$+\frac{b}{2}+e)$

$+e$

x_2

Taking Moments About "E"

$V_2 \cos\theta(b-x_2)+V_1\cos\theta(b-x_1)=\Sigma V\cos\theta(\frac{b}{2}-e)$

$V_1 = \Sigma V - V_2$ (Vertical Equilibrium)

$V_2(b-x_2)+(\Sigma V-V_2)(b-x_1) = \Sigma V(\frac{b}{2}-e)$

$\cancel{V_2 b}-V_2x_2+\Sigma Vb-\Sigma Vx_1-\cancel{V_2 b}+V_2x_1=\Sigma V\frac{b}{2}-\Sigma Ve$

$V_2(x_1-x_2)=\Sigma V(x_1-b+\frac{b}{2}-e)$

$$V_2 = \Sigma V\left[\frac{x_1-\frac{b}{2}-e}{x_1-x_2}\right]$$

...ε

...$\cos(\theta+\alpha_1)+\sin(\theta+\alpha_1)\cot\varepsilon]$

$+N_{H_1}$

...$)+\sin(\theta+\alpha_1)\cot\varepsilon]\tan\phi$

...$)+\sin(\theta+\alpha_2)\cot\varepsilon]\tan\phi$

...$\theta+\alpha_1)$

...$\theta+\alpha_2)$

$H_1\cos(\theta+\alpha_1)=F_{f_1}+F_{G_1}$, And $H_2\cos(\theta+\alpha_2)=F_{f_2}+F_{G_2}$ -- For Static Equilibrium Of Free Body Diagrams

$H_1\cos(\theta+\alpha_1) = V_1[\cos(\theta+\alpha_1)+\sin(\theta+\alpha_1)\cot\varepsilon]\tan\phi + V_1\sin(\theta+\alpha_1)$

$$H_1 = V_1\frac{[\cos(\theta+\alpha_1)+\sin(\theta+\alpha_1)\cot\varepsilon]\tan\phi+\sin(\theta+\alpha_1)}{\cos(\theta+\alpha_1)}$$

$$\therefore H_2 = V_2\frac{[\cos(\theta+\alpha_2)+\sin(\theta+\alpha_2)\cot\varepsilon]\tan\phi+\sin(\theta+\alpha_2)}{\cos(\theta+\alpha_2)}$$

$H_1+H_2=\Sigma H$ And $V_1+V_2=\Sigma V$ --- Static Equilibrium For Complete System

...fficients Of ΣV To Yield Net Horizontal Resistance To Sliding Downstream

...Due To ...ric Loading @ α_1

Coefficient Due To Friction @ Undulation α_1

Coefficient Due To Gravity @ Undulation α_1

Coefficient Due To Eccentric Loading @ α_2

Coefficient Due To Friction @ Undulation α_2

Coefficient Due To Gravity @ α_2

$$\ldots]\frac{[\cos(\theta+\alpha_1)+\sin(\theta+\alpha_1)\cot\varepsilon]\tan\phi+[\sin(\theta+\alpha_1)]}{\cos(\theta+\alpha_1)} + \Sigma V\left[\frac{x_1-\frac{b}{2}-e}{x_1-x_2}\right]\frac{[\cos(\theta+\alpha_2)+\sin(\theta+\alpha_2)\cot\varepsilon]\tan\phi+\sin(\theta+\alpha_2)}{\cos(\theta+\alpha_2)}$$

$$\frac{\left[\frac{(-2x_2+b+2e)\left[[1+\tan(\theta+\alpha_1)\cot\varepsilon]\tan\phi+\tan(\theta+\alpha_1)\right]+(2x_1-b-2e)\left[[1+\tan(\theta+\alpha_2)\cot\varepsilon]\tan\phi+\tan(\theta+\alpha_2)\right]}{2(x_1-x_2)}\right]}{\Sigma H}$$

...α Of $(\theta+\alpha)$ Are Obtained By Stereo-Net Plot Of Angles ...Hole Photography, Then :

$$\frac{\ldots 1+\tan(\theta+\alpha)\cot\varepsilon)\tan\phi+\tan(\theta+\alpha)\,]}{\Sigma H}$$

SYMBOL	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: M.A. Dixon
DRAWN: W.G. Binkley
CHECKED: P. Knowles
PREPARED:

SNETTISHAM PROJECT, ALASKA

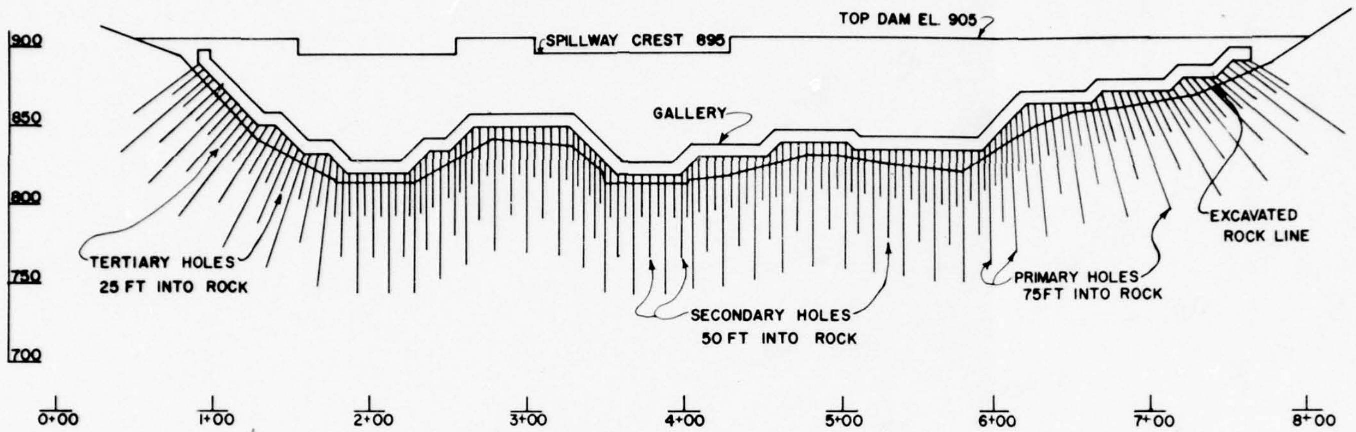
ANALYSIS OF SLIDING RESISTANCE

GENERAL CASE: A Low-angled Undulating Joint Surface Which "Daylights" in Upstream and Downstream Directions.

SUPERVISED: M.A. Dixon, GEOLOGY SECTION
APPROVED: Clare J. Farley
SUBMITTED: ... FOUNDATIONS AND MATERIALS BRANCH
RECOMMENDED: ...
SCALE | DATE 25 Oct 65 | FILE NUMBER
SHEET OF

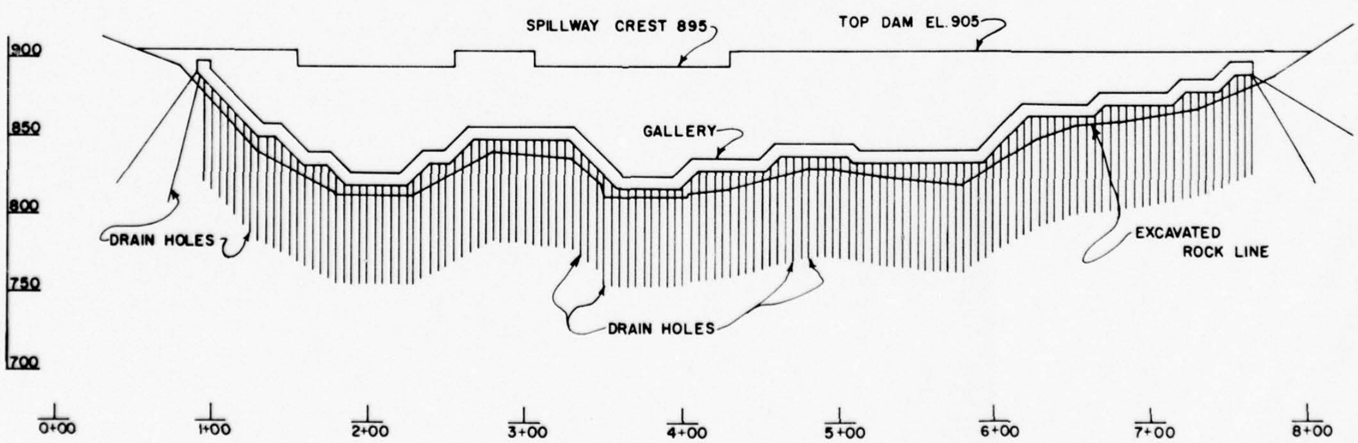
DESIGN MEMORANDUM NO. 7 PLATE 14-A

2



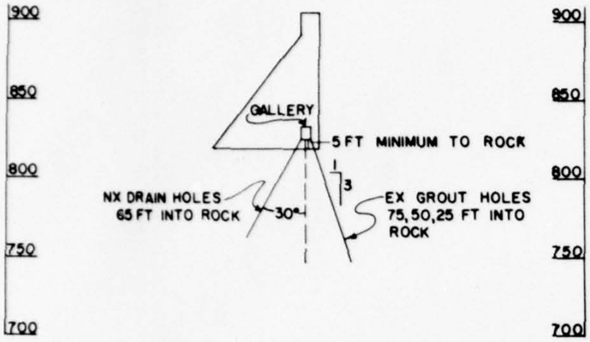
GROUTING DIAGRAM

UPSTREAM ELEVATION



DRAINAGE DIAGRAM

UPSTREAM ELEVATION



TYPICAL SECTION

GENERAL NOTES:

1. CURTAIN GROUTING TO BE DONE BY STAGE GROUTING METHODS
2. GROUT TAKE ESTIMATED AT 0.15 BAG CEMENT PER LINEAL FOOT OF EX GROUT HOLE.
3. CURTAIN GROUTING REQUIRED FOR PLAN B WILL BE ON A GREATLY REDUCED SCALE BECAUSE OF EXTENSIVE AREA GROUTING OVER FOUNDATION.
4. DRAIN HOLES ON PLAN B WILL BE ON 4 FT CENTERS INSTEAD OF 5 FT BECAUSE OF TIGHTER ANTICIPATED FOUNDATION DUE TO AREA GROUTING

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: Wm. Trautwein
DRAWN: Wm. Trautwein
CHECKED: D. Hanson
PREPARED:
CHIEF

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM SITE – PLAN A
FOUNDATION GROUTING AND DRAINAGE

SUPERVISED: N. A. Dixon
CHIEF GEOLOGY SECTION

APPROVED: Clare J. Farley
COLL. NEL, CE, DISTRICT ENGINEER

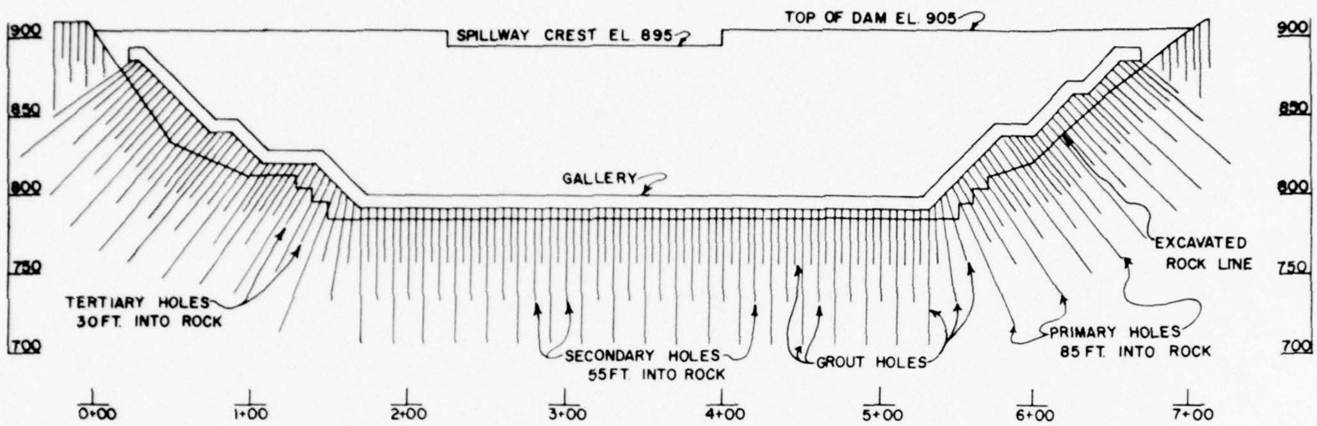
SUBMITTED: [signature]
CHIEF FOUNDATIONS & MATERIALS BRANCH

SCALE
DATE 20 Oct 65

RECOMMENDED: Warren George
CHIEF ENGINEERING DIVISION

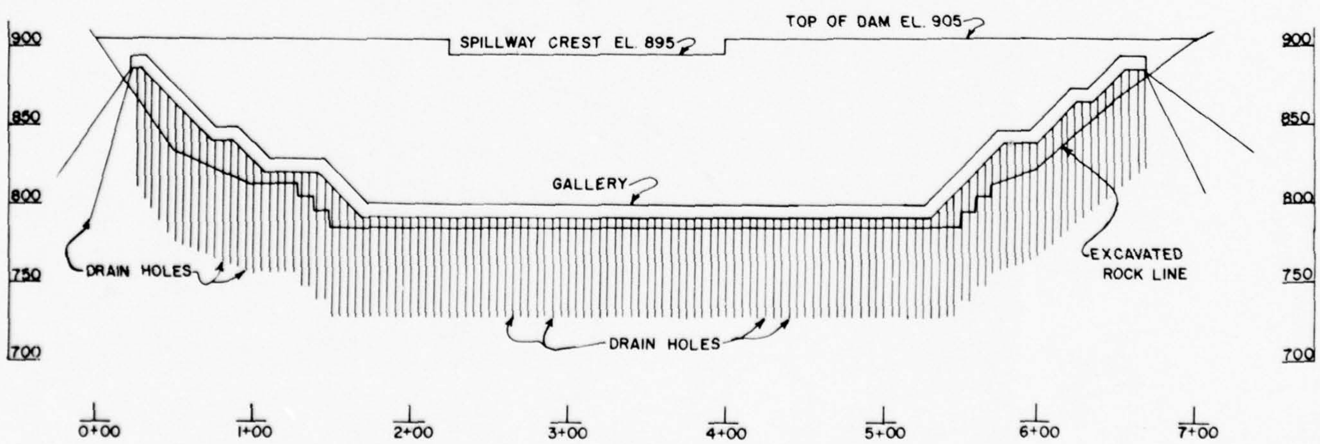
FILE NUMBER

SHEET OF



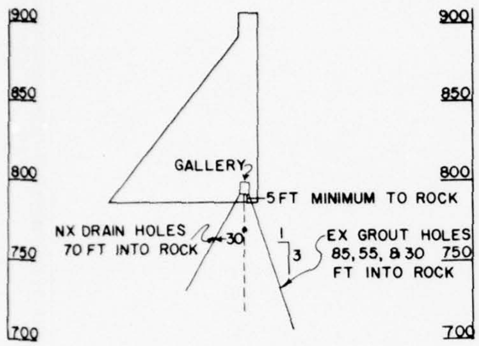
GROUTING DIAGRAM

UPSTREAM ELEVATION



DRAINAGE DIAGRAM

UPSTREAM ELEVATION



TYPICAL SECTION

GENERAL NOTES:

1. CURTAIN GROUTING TO BE DONE BY STAGE GROUTING METHODS
2. GROUT TAKE ESTIMATED AT 0.15 BAG OF CEMENT PER LINEAL FOOT OF EX GROUT HOLE

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: WM. TRAUTWEIN
DRAWN: WM. TRAUTWEIN
CHECKED: D. HANSEN
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM SITE-PLAN C
FOUNDATION GROUTING
AND DRAINAGE

SUPERVISED: W. A. Dixon — CHIEF, GEOLOGY SECTION
SUBMITTED: CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED: Warren George — CHIEF, ENGINEERING DIVISION
APPROVED: COLONEL, CE, DISTRICT ENGINEER
SCALE
DATE 20 Oct 65
FILE NUMBER
SHEET OF

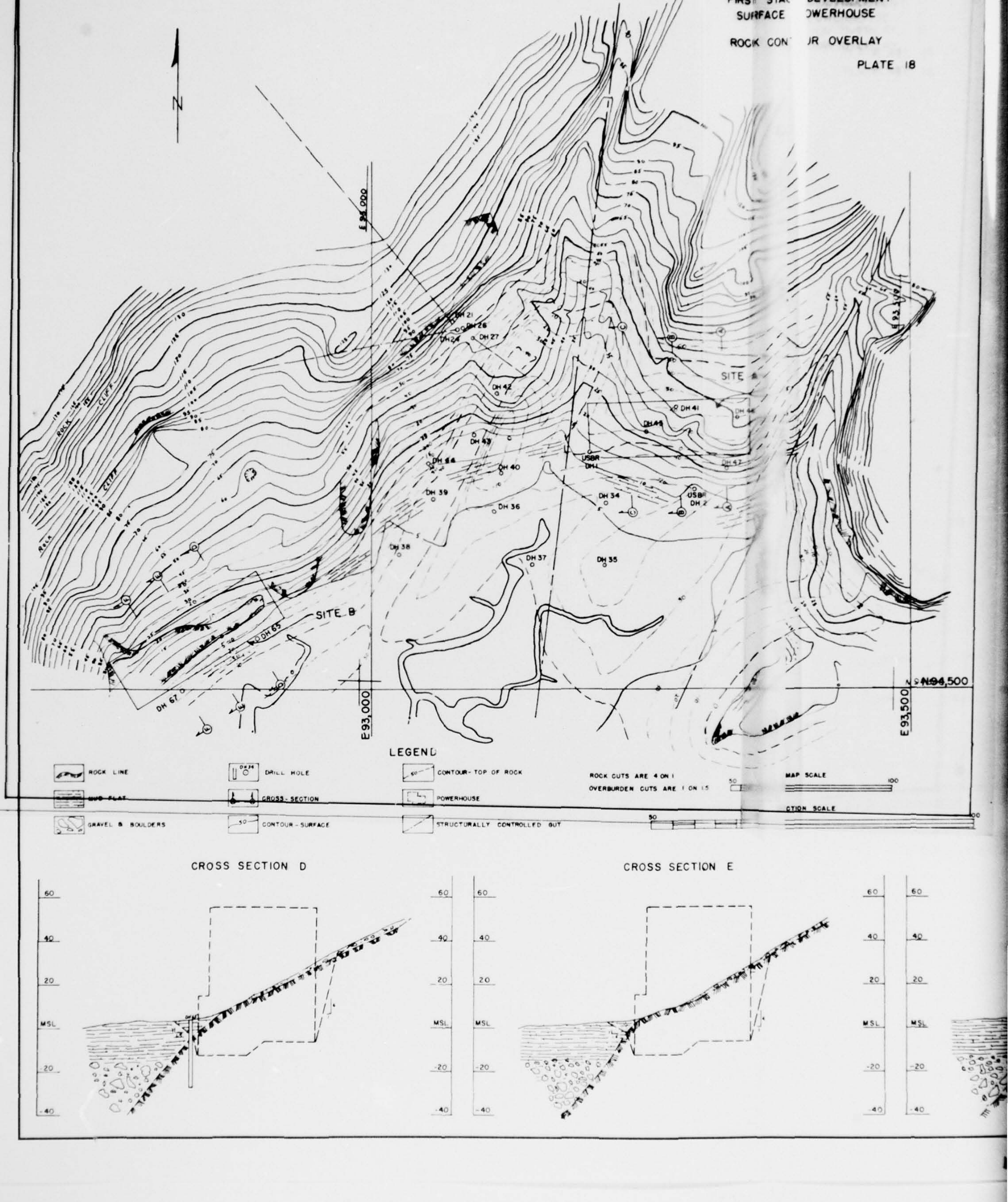
2

CORPS OF ENGINEERS

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
SURFACE POWERHOUSE

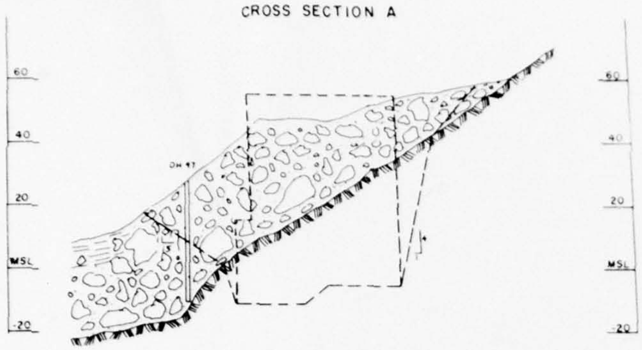
ROCK CONTOUR OVERLAY

PLATE 18

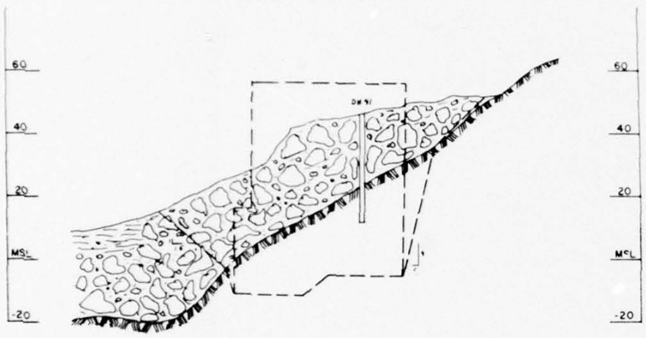


U. S. ARMY

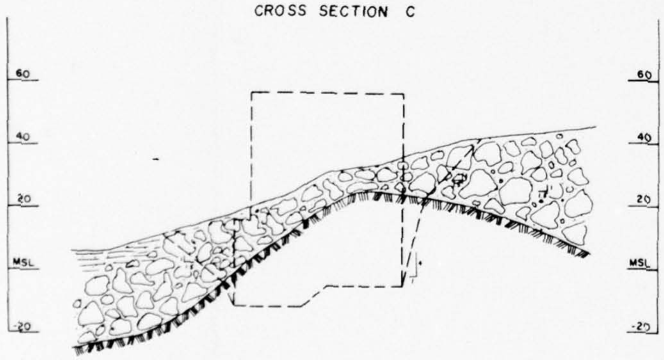
CROSS SECTION A



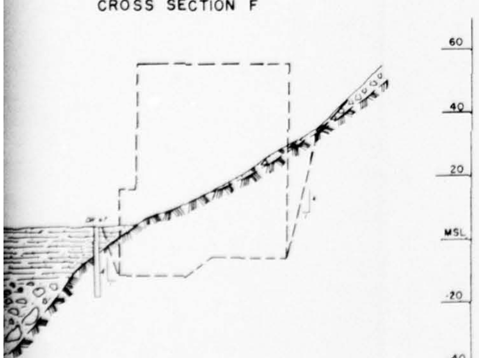
CROSS SECTION B



CROSS SECTION C



CROSS SECTION F



REVISION DATE DESCRIPTION BY

U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

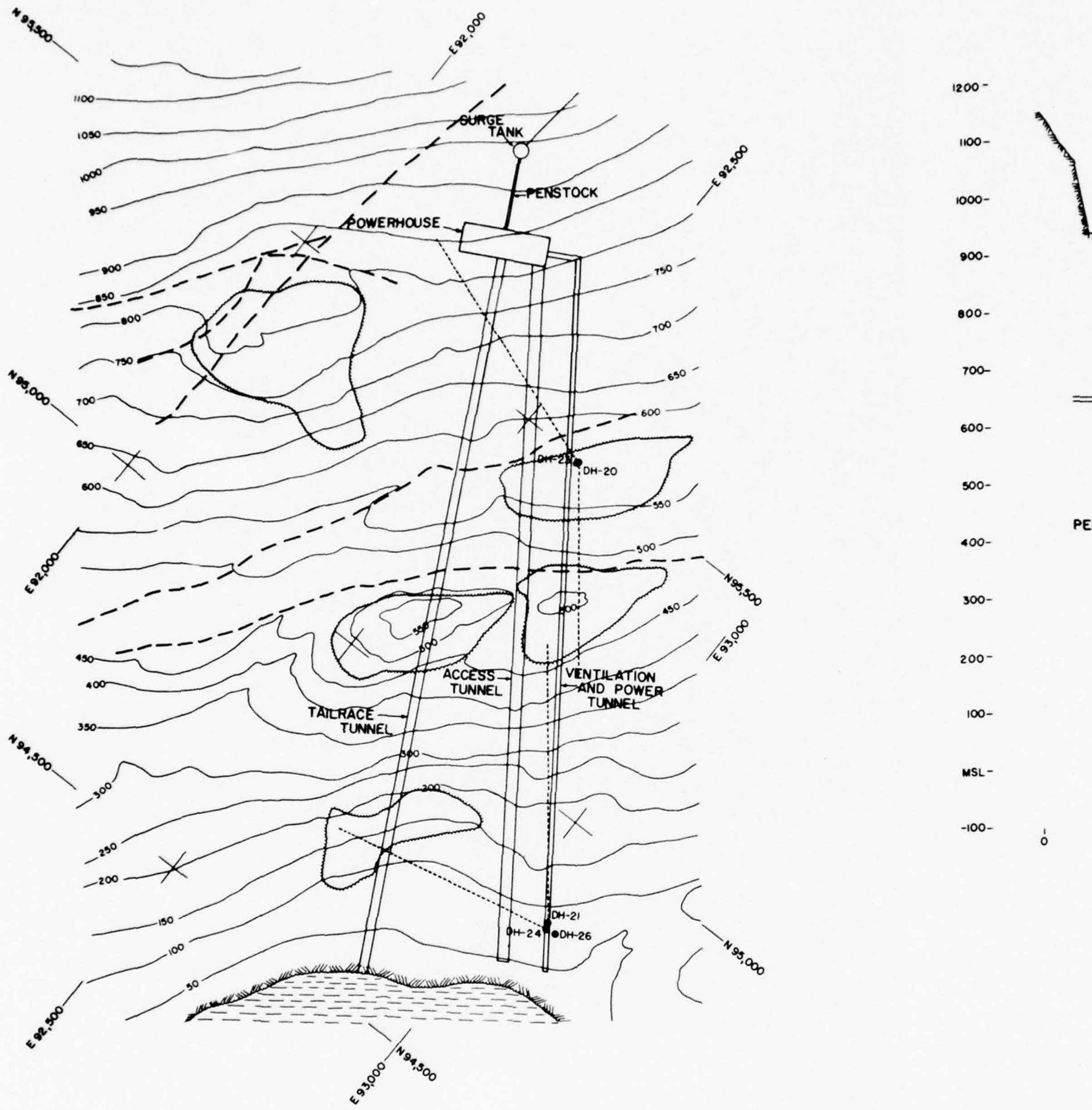
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

SURFACE POWERHOUSE
SITING STUDIES

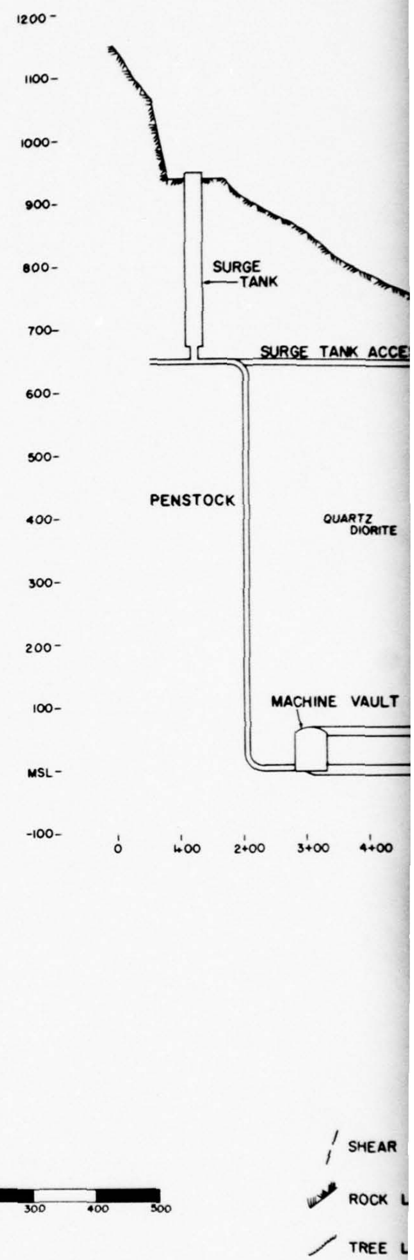
DESIGNED: Wm. Trautwein	
DRAWN: Wm. Trautwein	
CHECKED:	
PREPARED:	
SUPERVISED: CHIEF GEOLOGY SECTION	APPROVED: COLONEL, CE, DISTRICT ENGINEER
SUBMITTED: CHIEF FOUNDATIONS & MATERIALS BRANCH	SCALE · DATE 20 Oct 65
RECOMMENDED: CHIEF ENGINEERING DIVISION	FILE NUMBER · SHEET OF

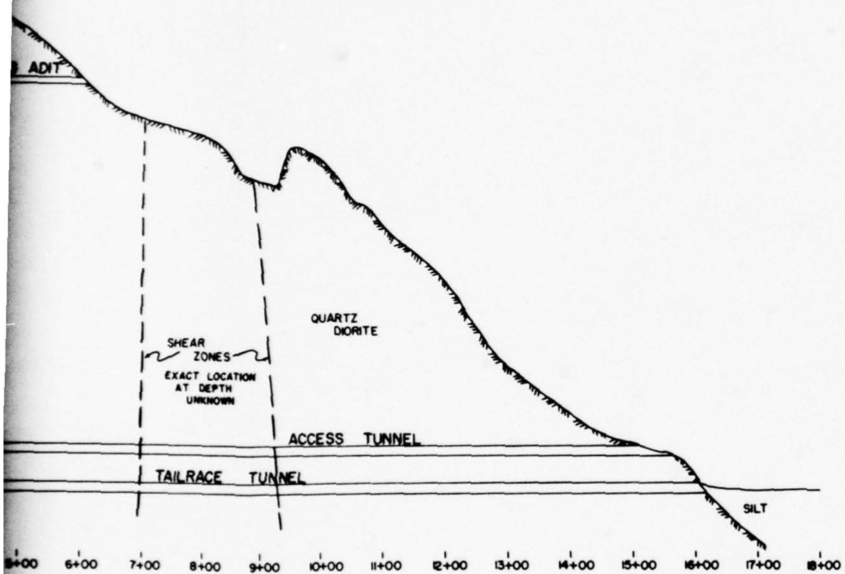
2

CORPS OF ENGINEERS



PLAN VIEW





SECTIONAL VIEW

ONE

E

E

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

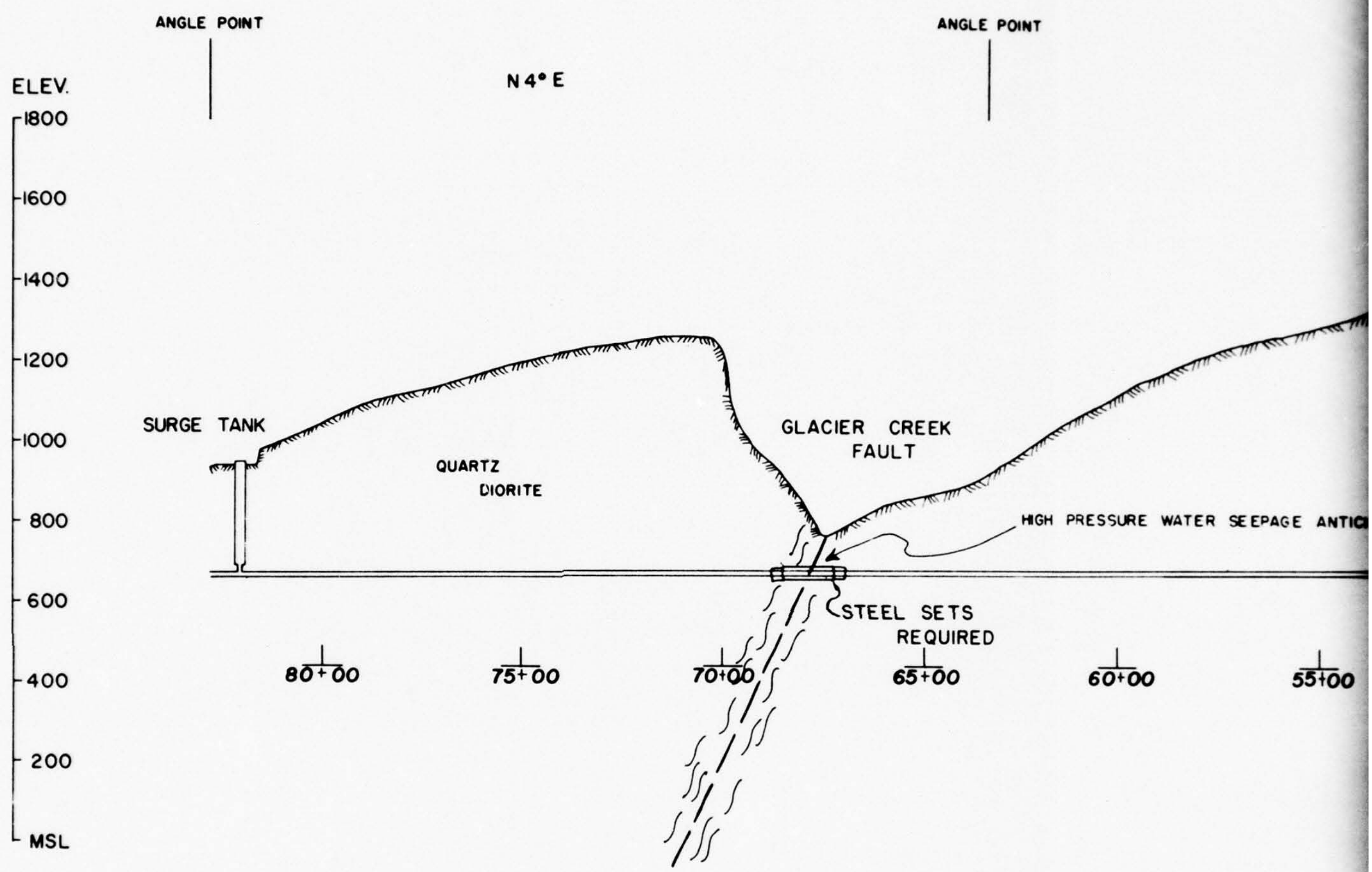
DESIGNED: D. Hansen
DRAWN: D. Hansen
CHECKED: W. E. Binkley
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
UNDERGROUND POWERHOUSE
SITE STUDY

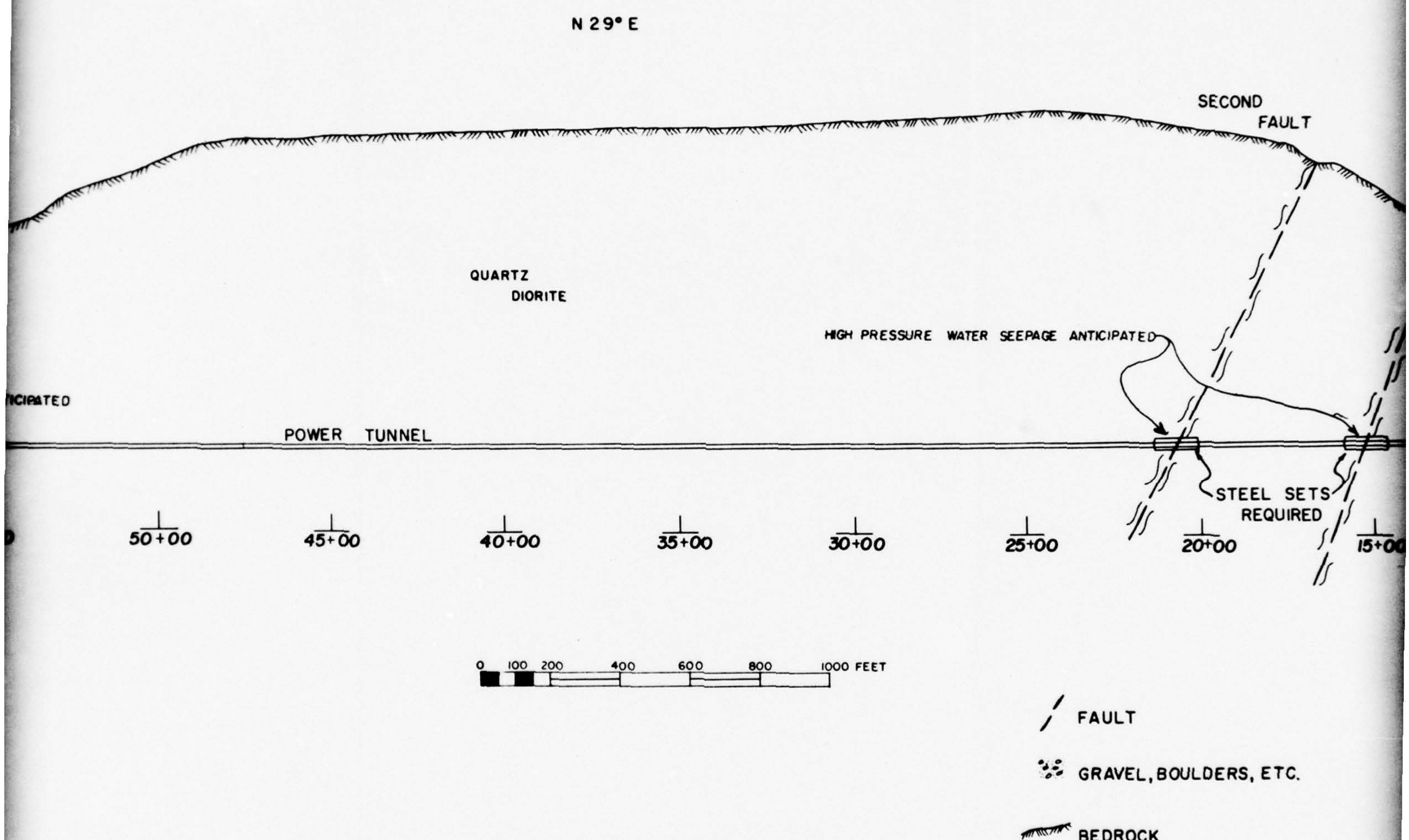
SUPERVISED: CHIEF, GEOLOGY SECTION
SUBMITTED: CHIEF, FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED: CHIEF, ENGINEERING DIVISION
APPROVED: COLONEL, CE, DISTRICT ENGINEER
SCALE
DATE 20 Oct 65
FILE NUMBER
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 19

2


ANGLE POINT
ANGLE POINT
N 4° E
ELEV.
1800
1600
1400
1200
1000
800
600
400
200
MSL
SURGE TANK
QUARTZ
DIORITE
GLACIER CREEK
FAULT
HIGH PRESSURE WATER SEEPAGE ANTIC
STEEL SETS
REQUIRED
80+00
75+00
70+00
65+00
60+00
55+00

1327-66



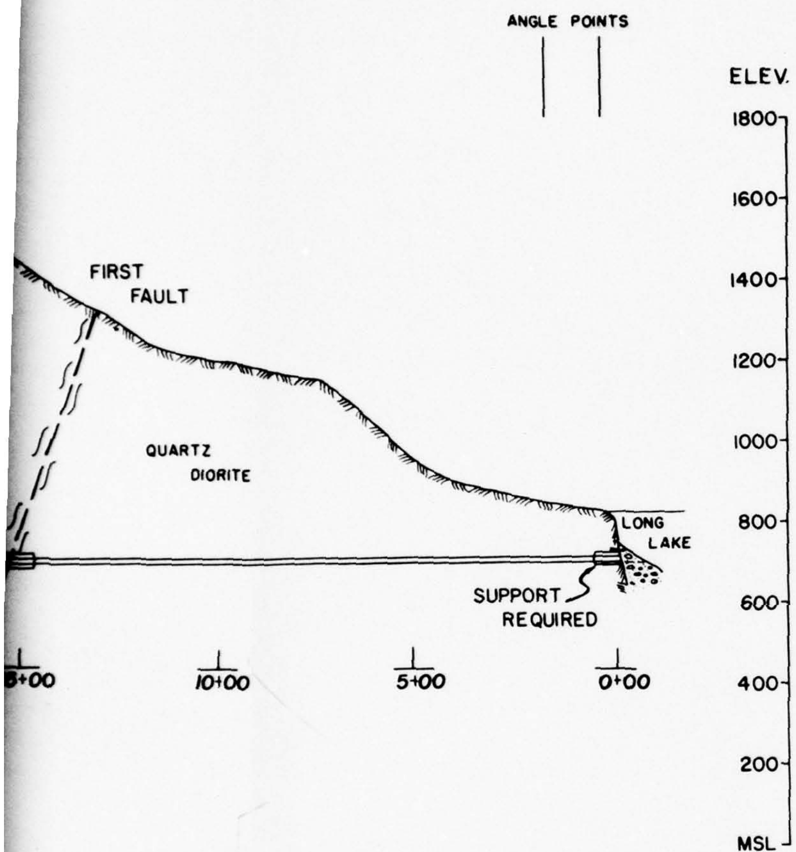
1 HARD COMPETENT QUARTZ DIORITE EXPECTED IN NEARLY ALL PORTIONS PENETRATED BY POWER TUNNEL, EXCEPT IN SPECIFIC AREAS OF FAULT ZONES.

2 A PATTERN OF ROCK BOLTS WILL BE PLACED FROM SPRING LINE TO SPRING LINE ACROSS ARCH OF POWER TUNNEL IMMEDIATELY FOLLOWING EXCAVATION THROUGHOUT LENGTH OF TUNNEL. THIS PROCEDURE WILL INSURE LESS OVERBREAK AND AFFORD GREATER SAFETY FROM ROCK FALL. (SEE SECTION 706 IN TEXT)

3. OVERBREAKAGE ESTIMATED AT 25 PERCENT WITH CONVENTIONAL METHODS OF TUNNELING.

4 SEE SECTIONS 708 & 711 IN TEXT FOR DISCUSSION OF DESIGN ROCK RESTRAINT AND STATE OF STRESS IN ROCK

2



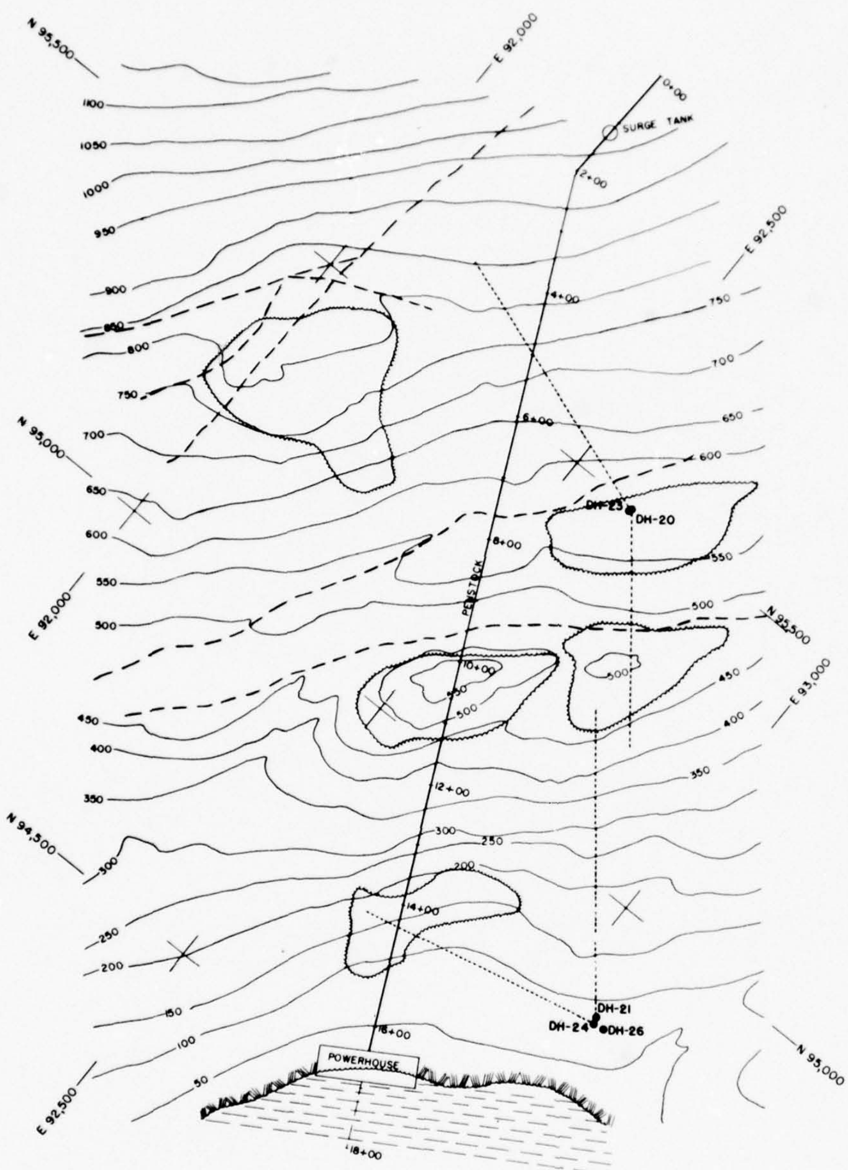
REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

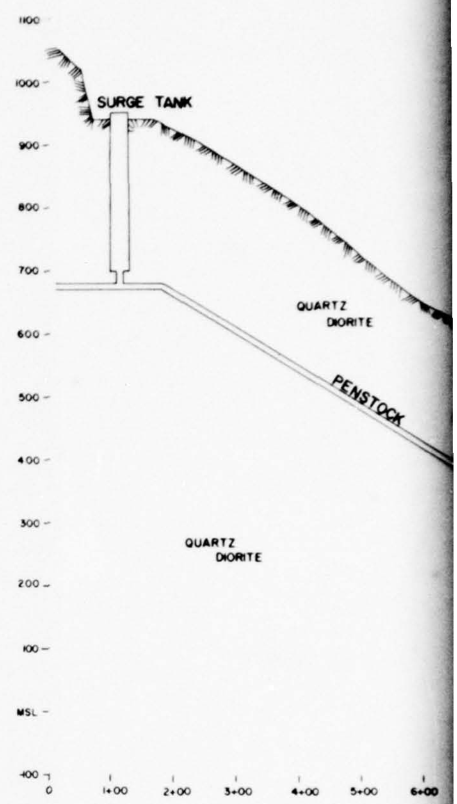
DESIGNED: D. Hansen	SNETTISHAM PROJECT, ALASKA FIRST STAGE DEVELOPMENT
DRAWN: W. Travison	
CHECKED: D J Lacher	
PREPARED: CHIEF,	POWER TUNNEL PROFILE
SUPERVISED: CHIEF, GEOLOGY SECTION	APPROVED: COLONEL, CE, DISTRICT ENGINEER
SUBMITTED: CHIEF, FOUNDATIONS & MATERIALS BRANCH	SCALE — DATE 20 Oct 65
RECOMMENDED: CHIEF, ENGINEERING DIVISION	FILE NUMBER SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 20

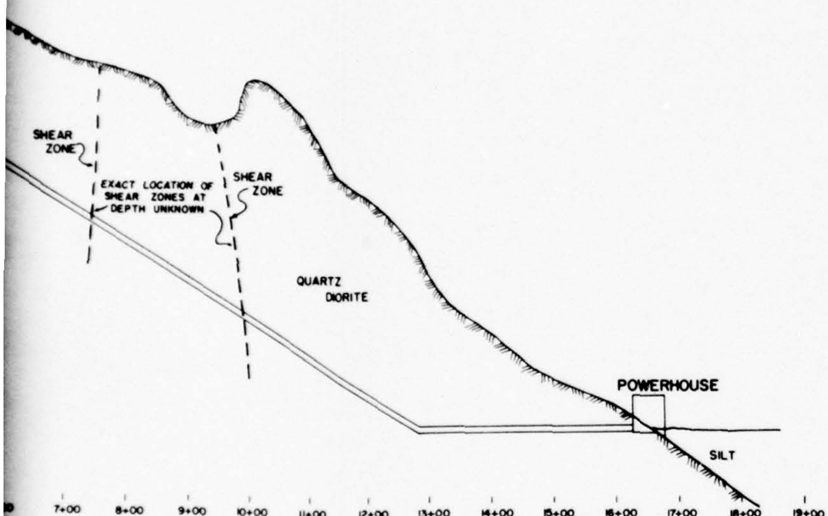
3


N 95,500
E 92,000
0+00
SURGE TANK
2+00
4+00
6+00
8+00
10+00
12+00
14+00
18+00
E 92,500
N 95,000
E 92,000
PENSTOCK
DH-23
DH-20
DH-21
DH-24
DH-26
N 95,500
E 93,000
N 94,500
POWERHOUSE
E 92,500
N 95,000
18+00
100
0
100
200
300
400
500

PLAN VIEW


1100
1000
900
800
700
600
500
400
300
200
100
MSL
-100
SURGE TANK
QUARTZ DIORITE
PENSTOCK
QUARTZ DIORITE
0
1+00
2+00
3+00
4+00
5+00
6+00

U. S. ARMY



SECTIONAL VIEW

SHEAR ZONE

ROCK LINE

TREE LINE

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

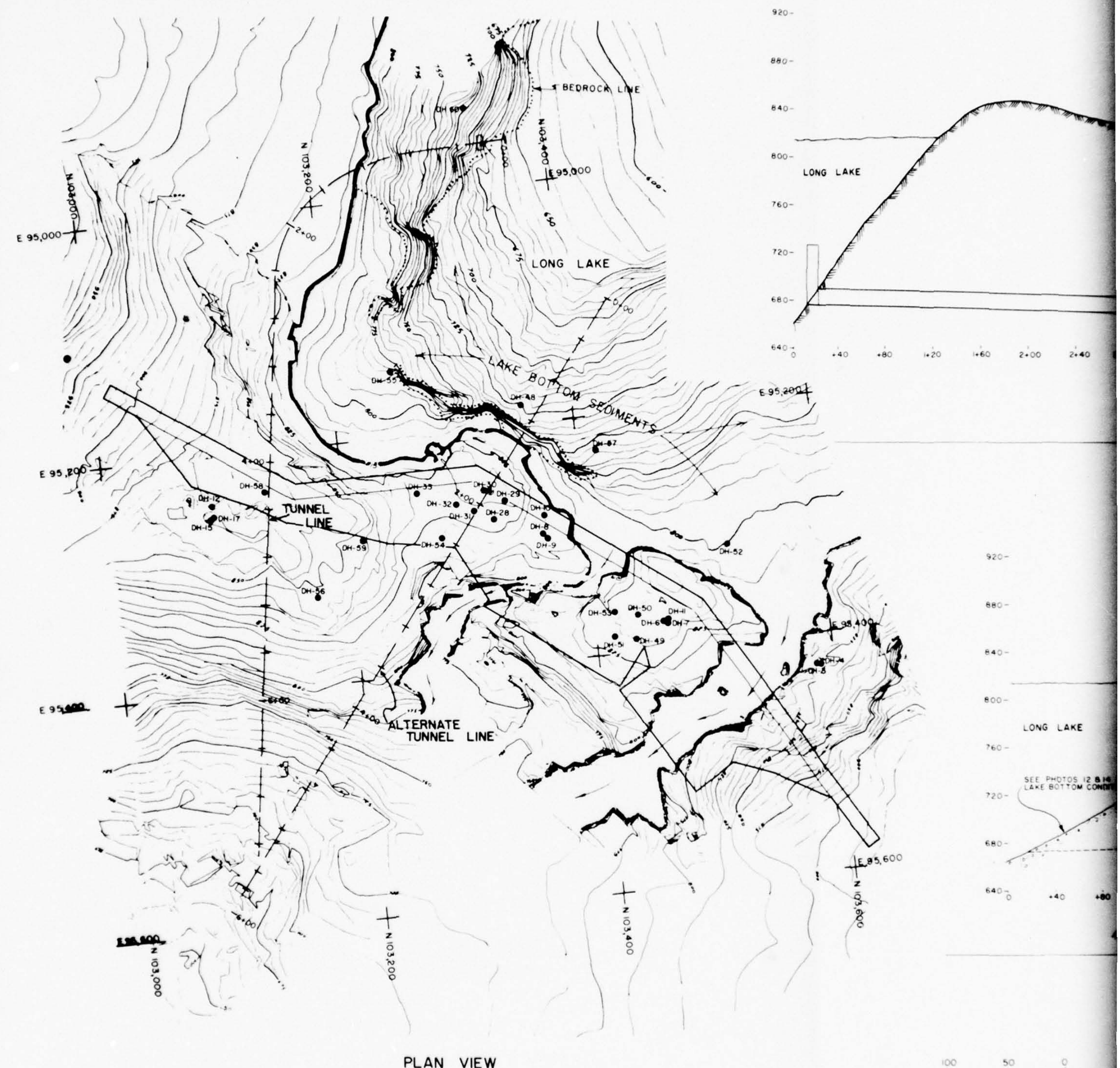
DESIGNED: D. Hansen
DRAWN: D. Hansen
CHECKED: W.G. Bickley
PREPARED:
CHIEF

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
SURFACE POWERHOUSE
PENSTOCK

SUPERVISED: M.A. Dixon
CHIEF, GEOLOGY SECTION
SUBMITTED:
CHIEF, FOUNDATION & MATERIALS BRANCH
RECOMMENDED: Warren George
CHIEF, ENGINEERING DIVISION

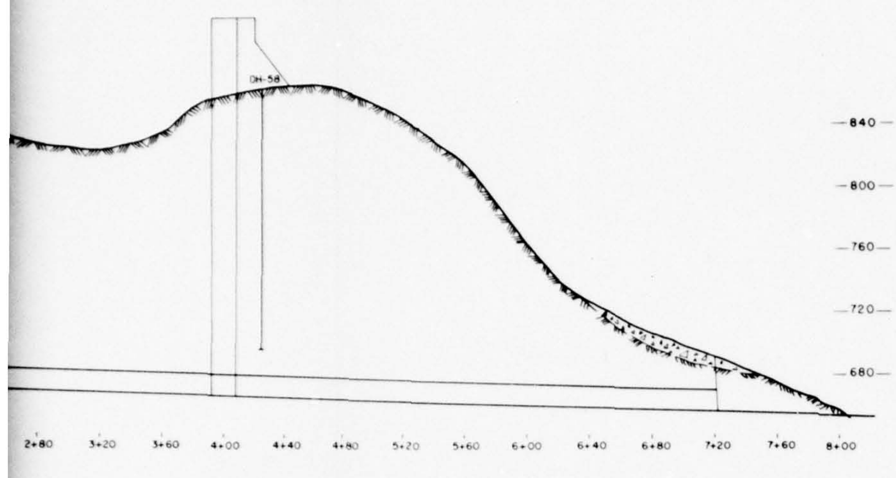
APPROVED:
COLONEL, CE, DISTRICT ENGINEER
SCALE
DATE 20 Oct 65
FILE NUMBER
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 21

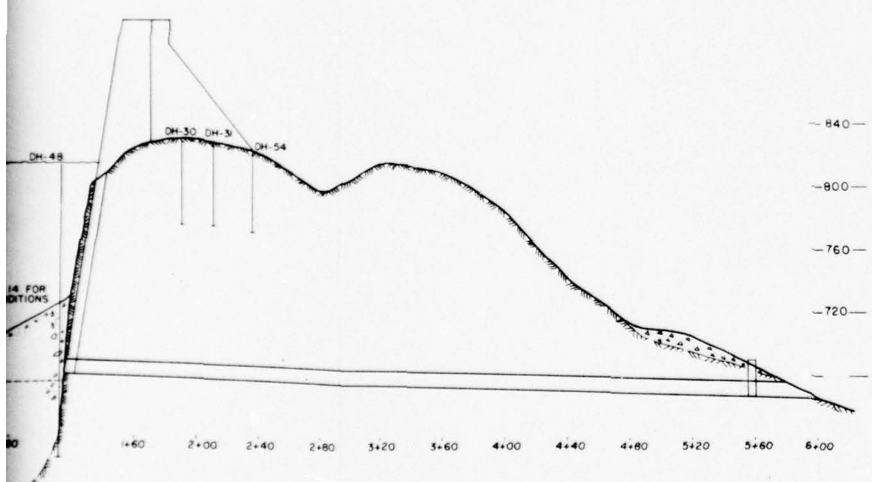

BEDROCK LINE
LONG LAKE
LAKE BOTTOM SEDIMENTS
TUNNEL LINE
ALTERNATE TUNNEL LINE
E 95,000
E 95,200
E 95,400
E 95,600
N 103,000
N 103,200
N 103,400
N 103,600
LONG LAKE
SEE PHOTOS 12 & 16
LAKE BOTTOM CONDI
SC

PLAN VIEW

U. S. ARMY



TUNNEL PROFILE



ALTERNATE TUNNEL PROFILE

100 200

SCALE OF FEET

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: D. Hansen
DRAWN: W.G. Binkley
CHECKED:
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

DIVERSION TUNNEL

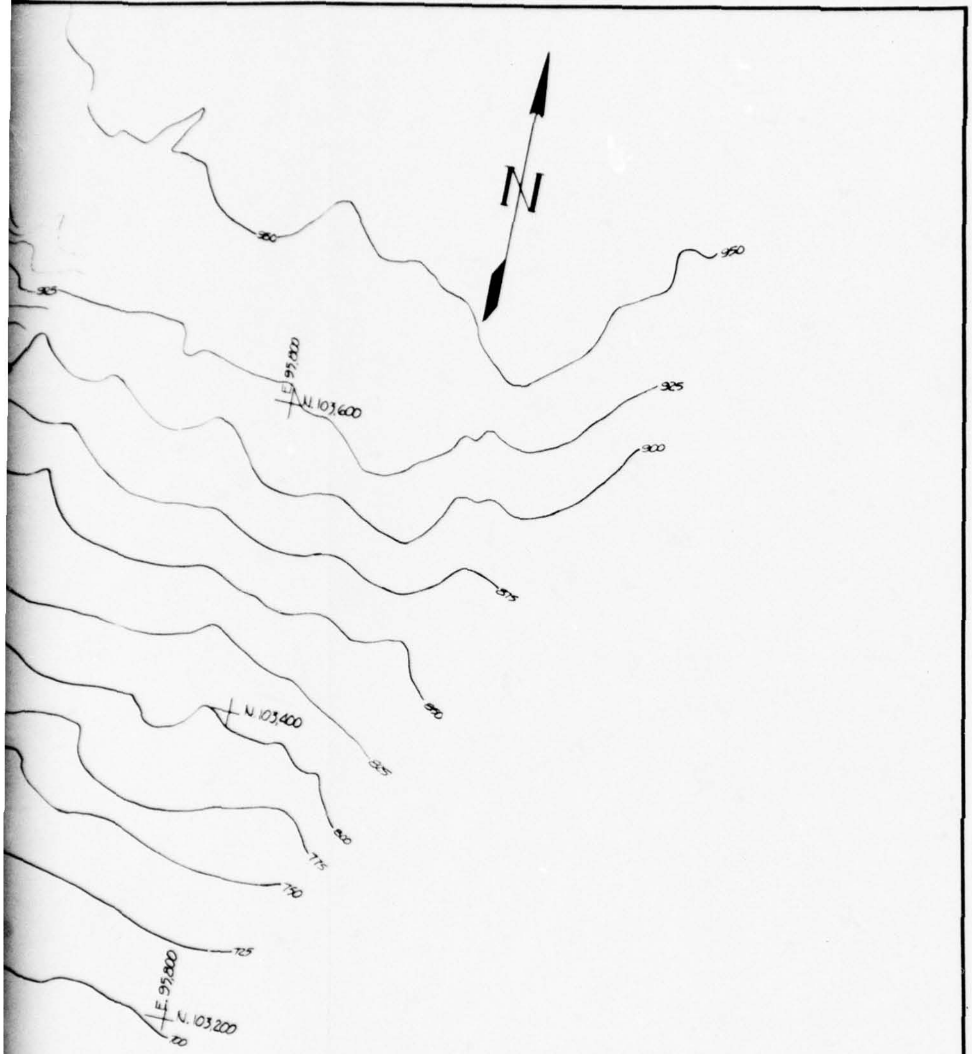
CHIEF
SUPERVISED: CHIEF GEOLOGY SECTION
SUBMITTED: CHIEF FOUNDATIONS & MATERIALS BRANCH
RECOMMENDED: CHIEF, ENGINEERING DIVISION

APPROVED: COLONEL, CE, DISTRICT ENGINEER

SCALE
DATE 20 Oct 66
FILE NUMBER
SHEET OF

CORPS OF ENGINEERS

U. S. ARMY



GRAPHIC SCALES

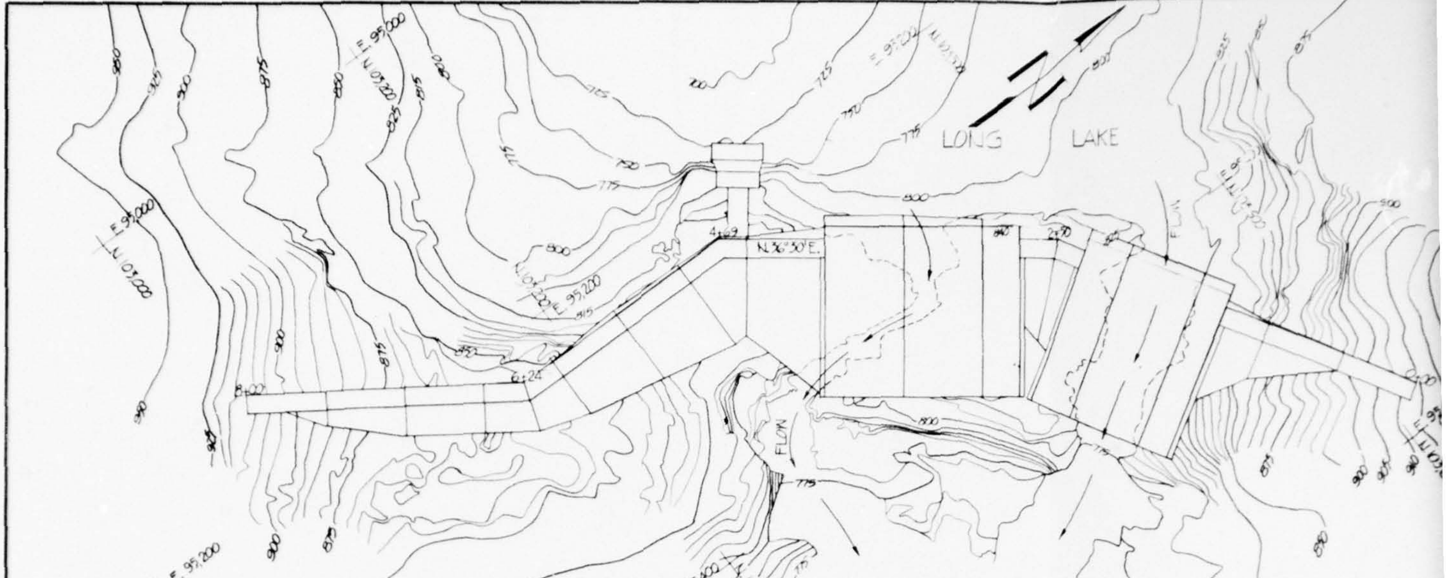
U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: J.P.P.
DRAWN:
CHECKED:
PREPARED:
SUPERVISED: Paul Knowles — CHIEF, STRUCTURAL SECTION
APPROVED: Clay I. Farley — COLONEL, CE DISTRICT ENGINEER
SUBMITTED: Arthur L. Jean — CHIEF, DESIGN BRANCH
RECOMMENDED: — CHIEF, ENGINEERING DIVISION

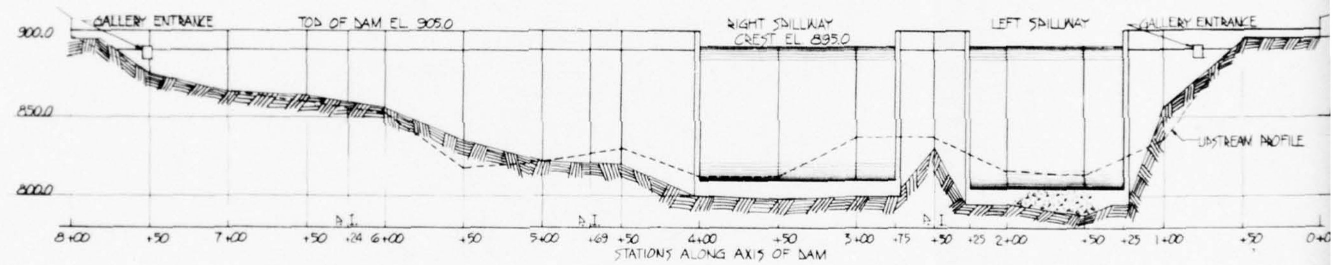
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM
PLAN "B"
SITE PLAN

SCALE 1" = 40'
DATE:
FILE NUMBER Q 4-5-113
SHEET OF

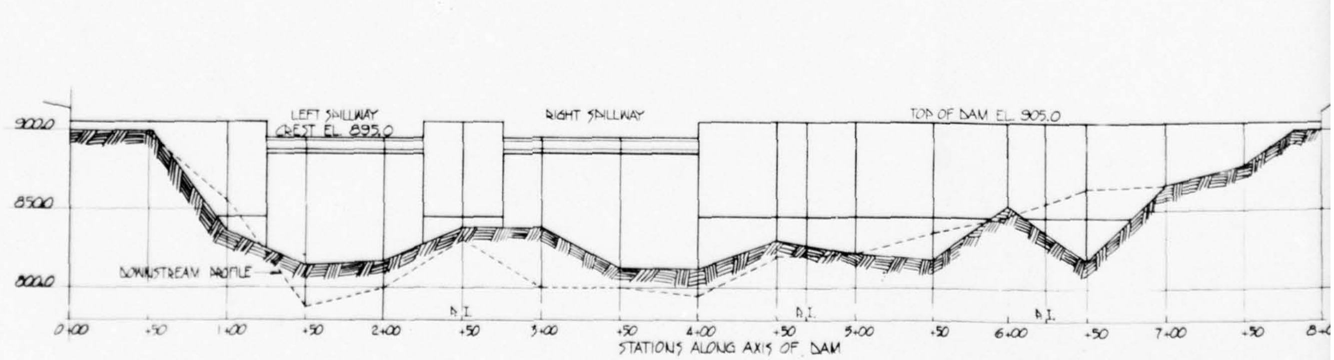
REV.	DATE	DWG. NO.	ACTION	DESCRIPTION	BY	APP'D



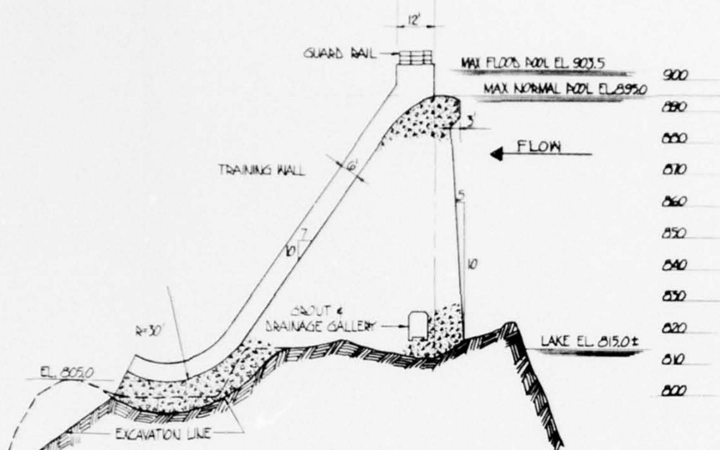
PLAN
GRAVITY DAM-BENT AXIS
SCALE: 1" = 40'



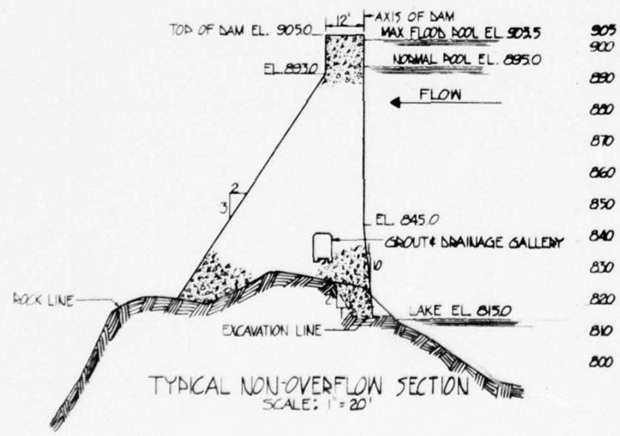
PROFILE
DOWNSTREAM LOOKING UPSTREAM
SCALE: 1" = 40



PROFILE
UPSTREAM LOOKING DOWNSTREAM
SCALE: 1" = 40'



TYPICAL SPILLWAY SECTION
SCALE: 1"= 20



TYPICAL NON-OVERFLOW SECTION
SCALE: 1"= 20'

REVISION	DATE	DESCRIPTION	BY

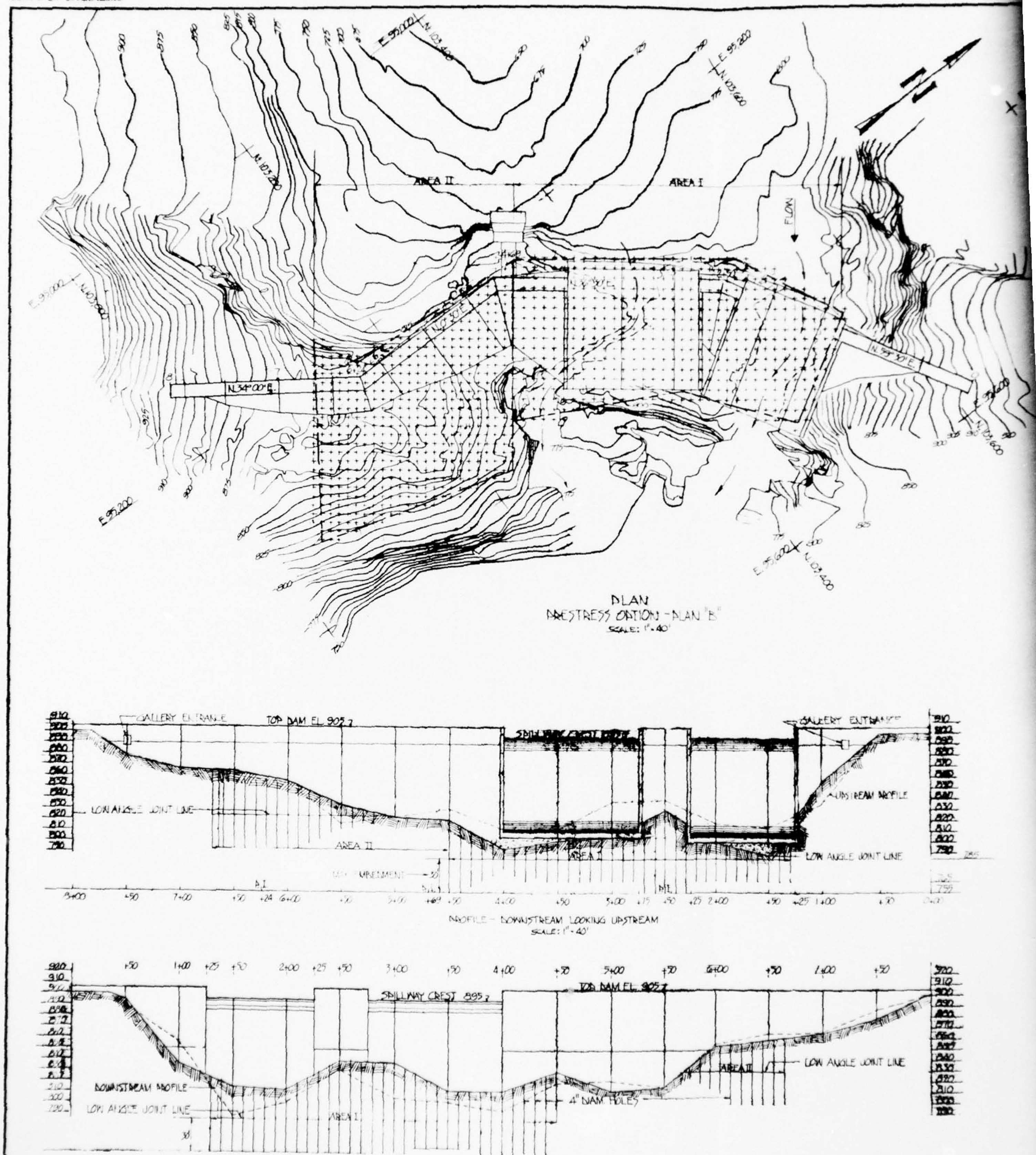
U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: J.P.D.
DRAWN: R.W.
CHECKED: R.C.W.
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM
PLAN "A"
GRAVITY DAM BENT AXIS

SUPERVISED: Paul Knowles, CHIEF, STRUCTURAL SECTION
SUBMITTED: Arthur L. Jones, CHIEF, DESIGN BRANCH
RECOMMENDED: Warren George, CHIEF, ENGINEERING DIVISION
APPROVED: COLONEL, CE, DISTRICT ENGINEER
DATE
SCALE AS- SHOWN
SPEC. NO.
DRAWING NUMBER Q 4-5-113
SHEET OF

GRAPHIC SCALES

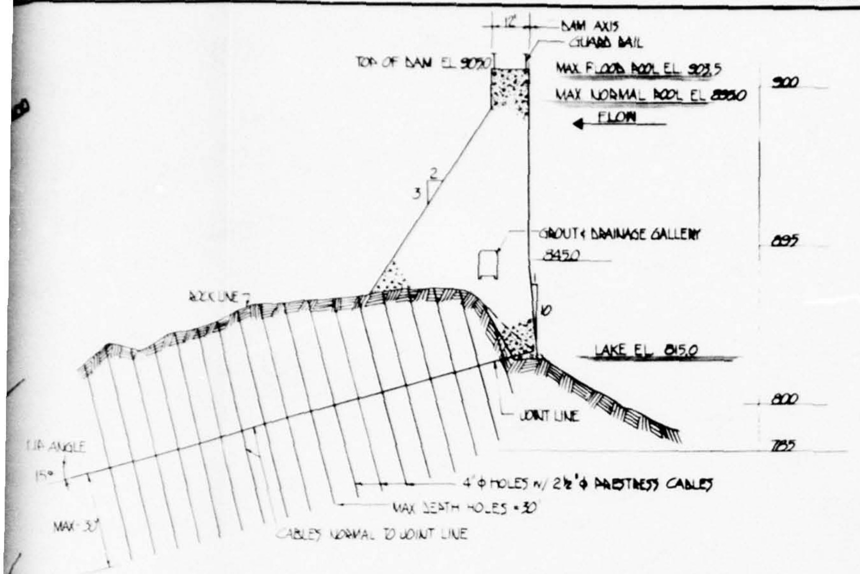


PLAN
PRESTRESS OPTION - PLAN "B"
SCALE: 1" = 40'

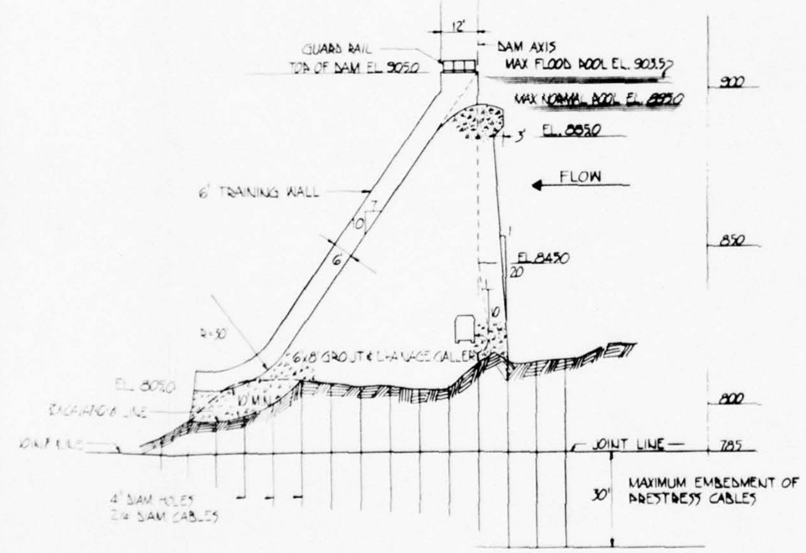
PROFILE - DOWNSTREAM LOOKING UPSTREAM
SCALE: 1" = 40'

PROFILE - UPSTREAM LOOKING DOWNSTREAM
SCALE: 1" = 40'

U. S. ARMY



AREA II - TYPICAL NON-OVERFLOW SECTION
SCALE: 1" = 20'



AREA I - TYPICAL SPILLWAY SECTION
SCALE: 1" = 20'

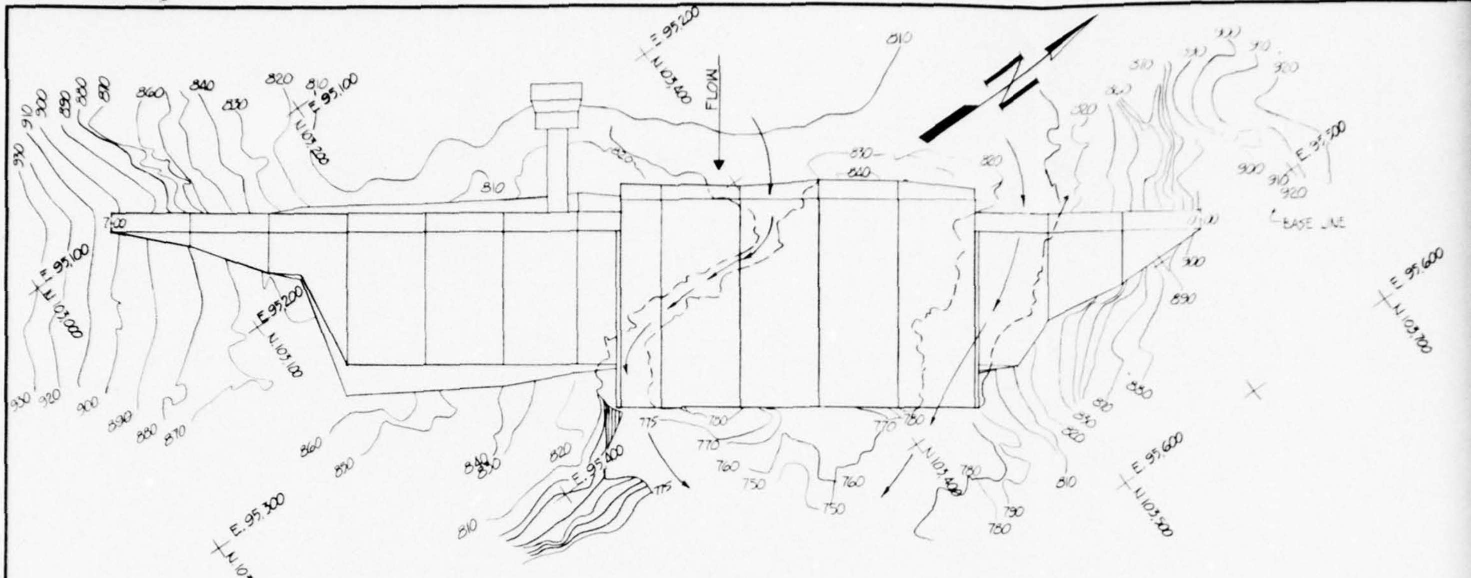
REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

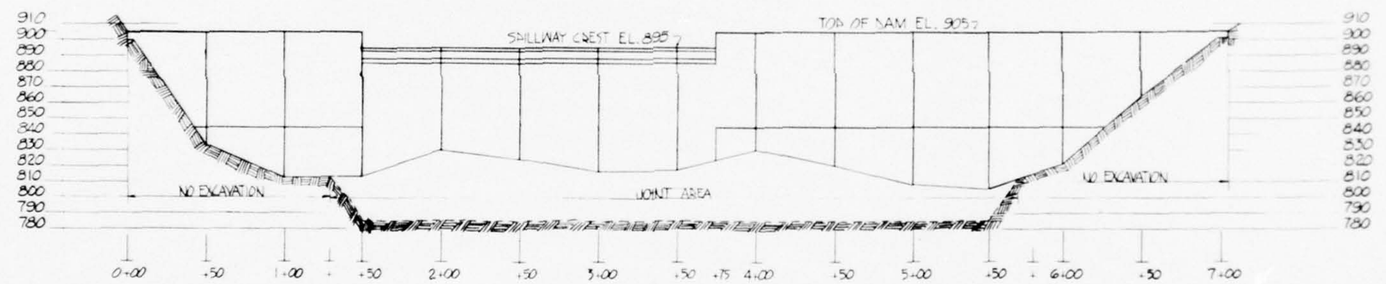
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM
PLAN "B"
GRAVITY DAM, PRESTRESS FDN. BENT AXIS

DESIGNED:
DRAWN:
CHECKED:
PREPARED:
SUPERVISED: Paul Knowles — CHIEF, STRUCTURAL SECTION
SUBMITTED: — CHIEF, DESIGN BRANCH
RECOMMENDED: Warren George — CHIEF, ENGINEERING DIVISION
APPROVED: — COLONEL, CE, DISTRICT ENGINEER
SCALE AS SHOWN
DATE 20 Oct. 65
FILE NUMBER Q 4-5-113
SHEET OF

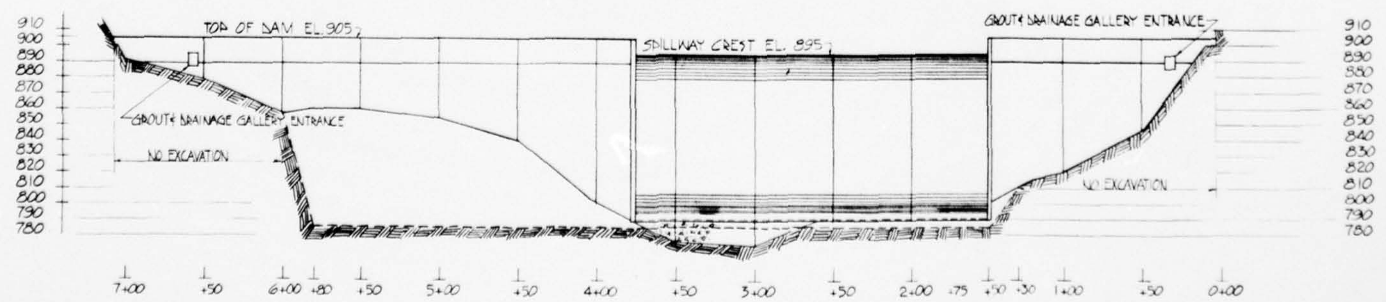
DESIGN MEMORANDUM NO. 7 PLATE NO. 25



PLAN
GRAVITY DAM - STRAIGHT AXIS
SCALE: 1" = 40'

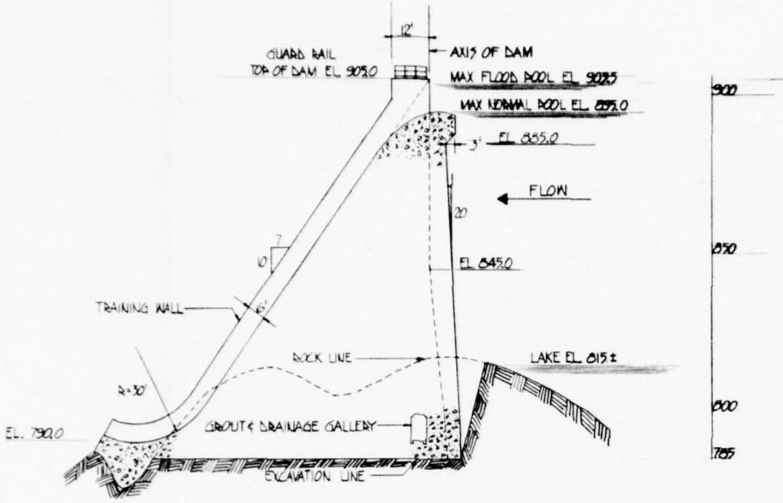


PROFILE
UPSTREAM LOOKING DOWNSTREAM
SCALE: 1" = 40'

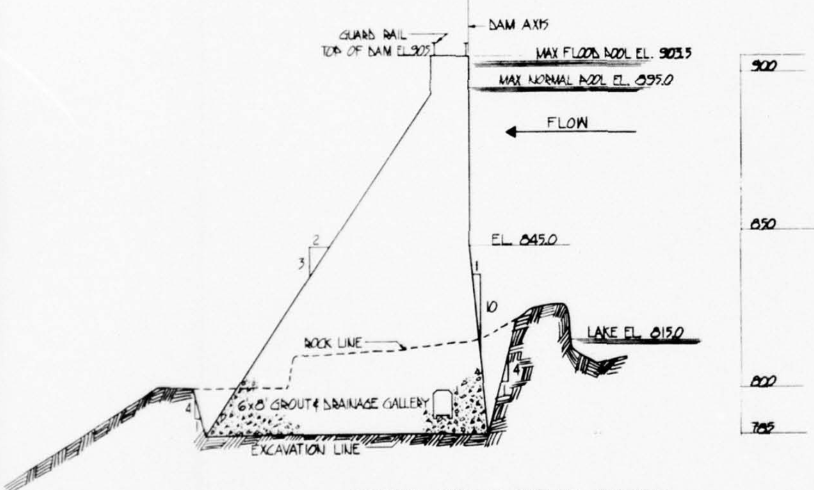


PROFILE
DOWNSTREAM LOOKING UPSTREAM
SCALE: 1" = 40'

U. S. ARMY



TYPICAL SPILLWAY SECTION
SCALE: 1"=20



TYPICAL NON-OVERFLOW SECTION
SCALE: 1"=20'

REV.	DATE	DWG. NO.	ACTION	DESCRIPTION	BY	APP'D.

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

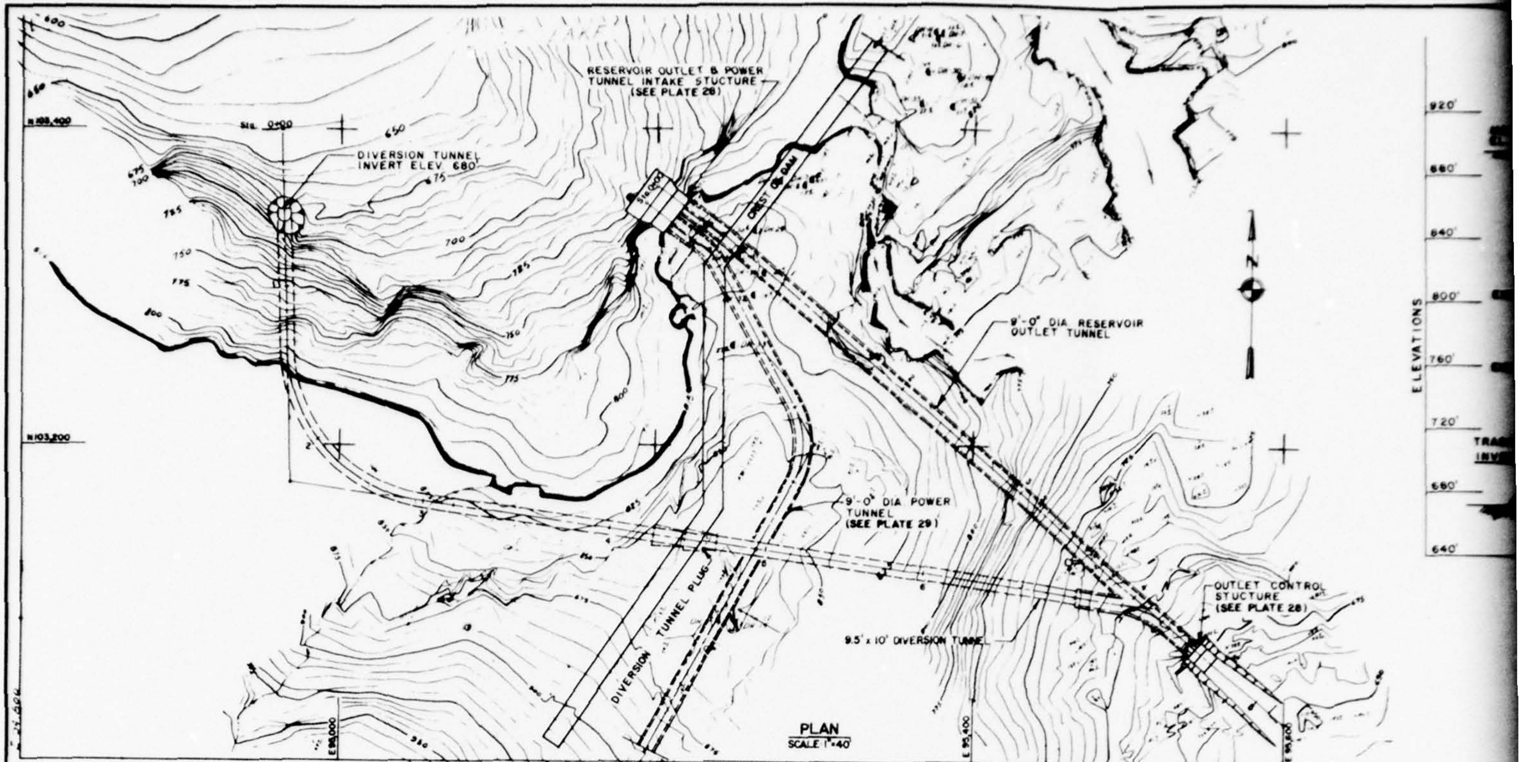
DESIGNED: J.P.P.
DRAWN: B.H.
CHECKED: N.W.
PREPARED: A. J. Greene, CHIEF, CIVIL UNIT

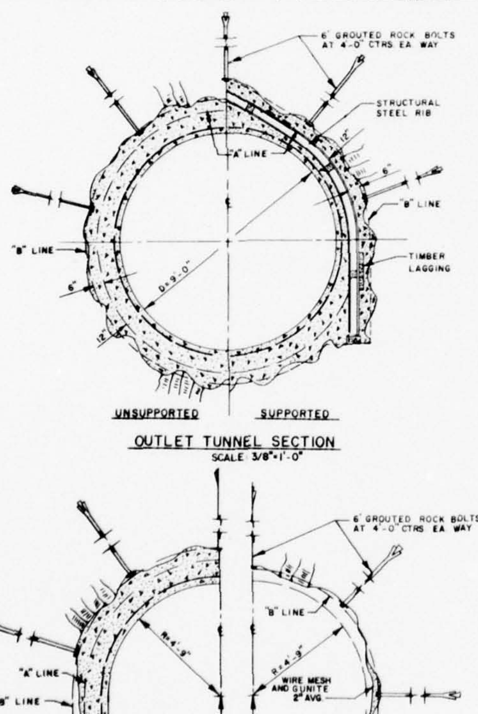
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
LONG LAKE DAM
PLAN "C"
GRAVITY SECTION STRAIGHT AXIS

SUPERVISE: Paul Knowles, CHIEF, STRUCTURAL SECTION
APPROVED: Clay D. Farley, COLONEL, CE, DISTRICT ENGINEER
DATE: 28 Oct. 65
SUBMITTED: William L. Jean, CHIEF, DESIGN BRANCH
SCALE: AS SHOWN
SPEC. NO.
RECOMMENDED: James Selinger, CHIEF, ENGINEERING DIVISION
DRAWING NUMBER: Q 4-5-113
SHEET OF

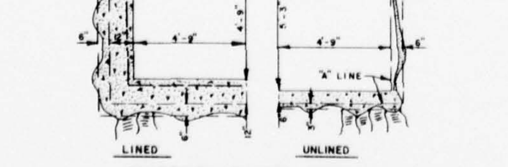
GRAPHIC SCALES

DESIGN MEMORANDUM NO. 7 PLATE NO. 26

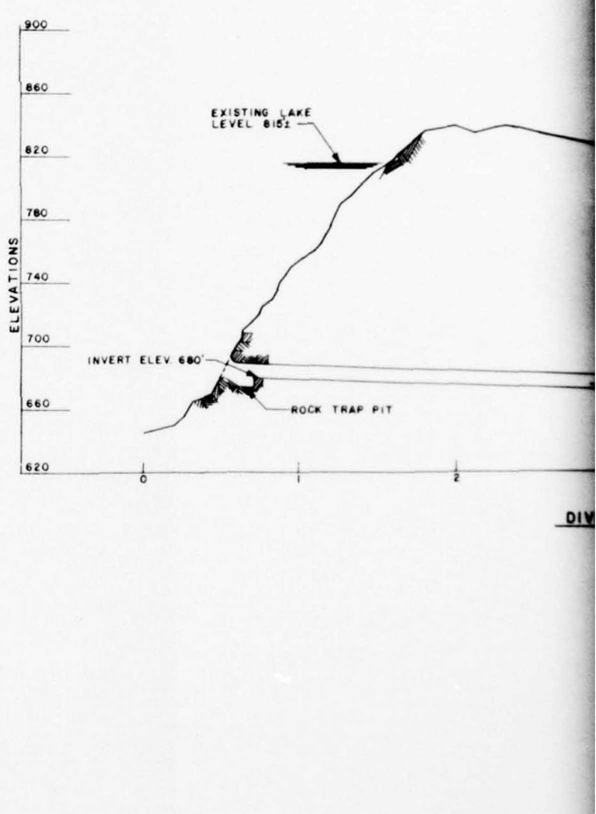


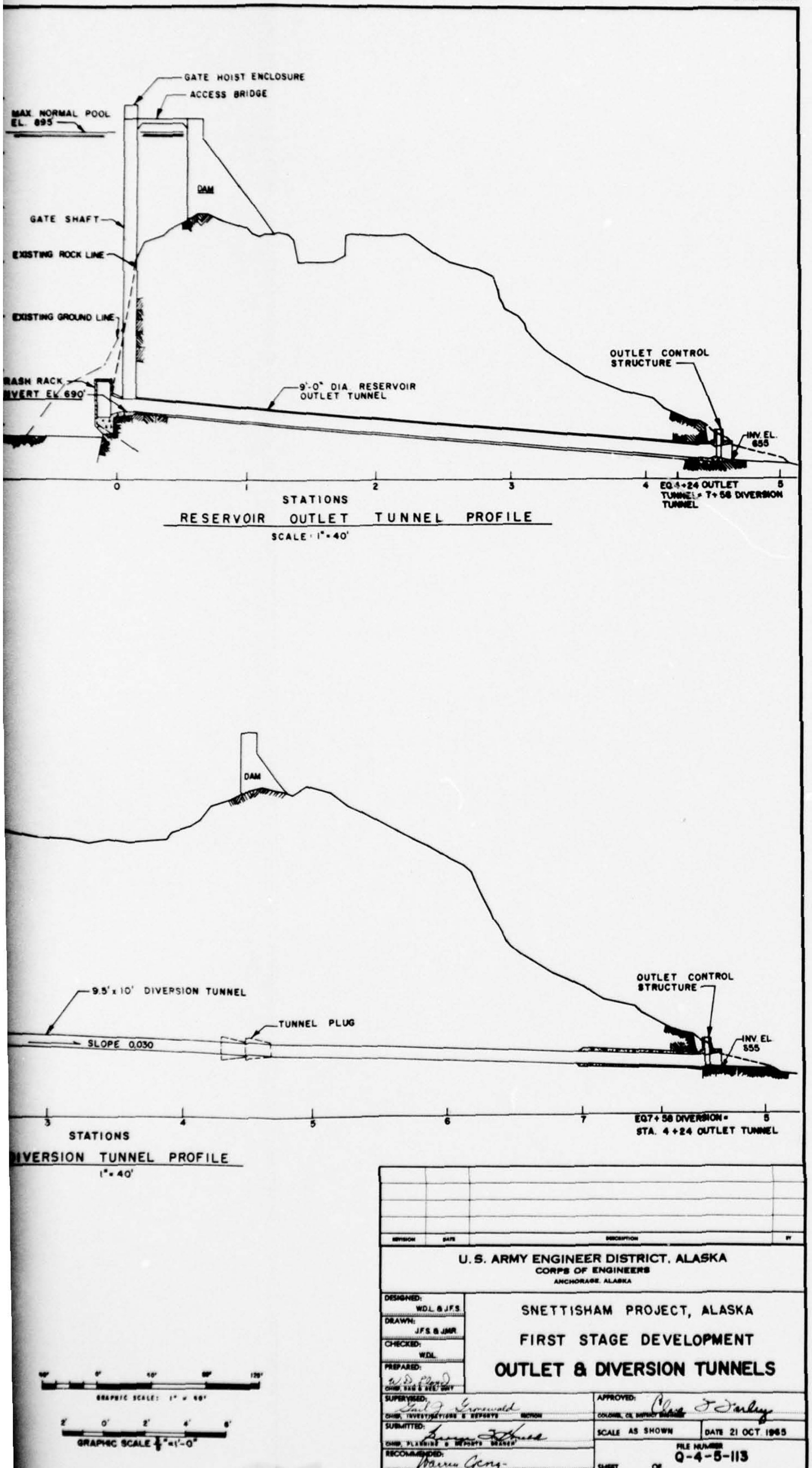


OUTLET TUNNEL SECTION
SCALE 3/8"=1'-0"

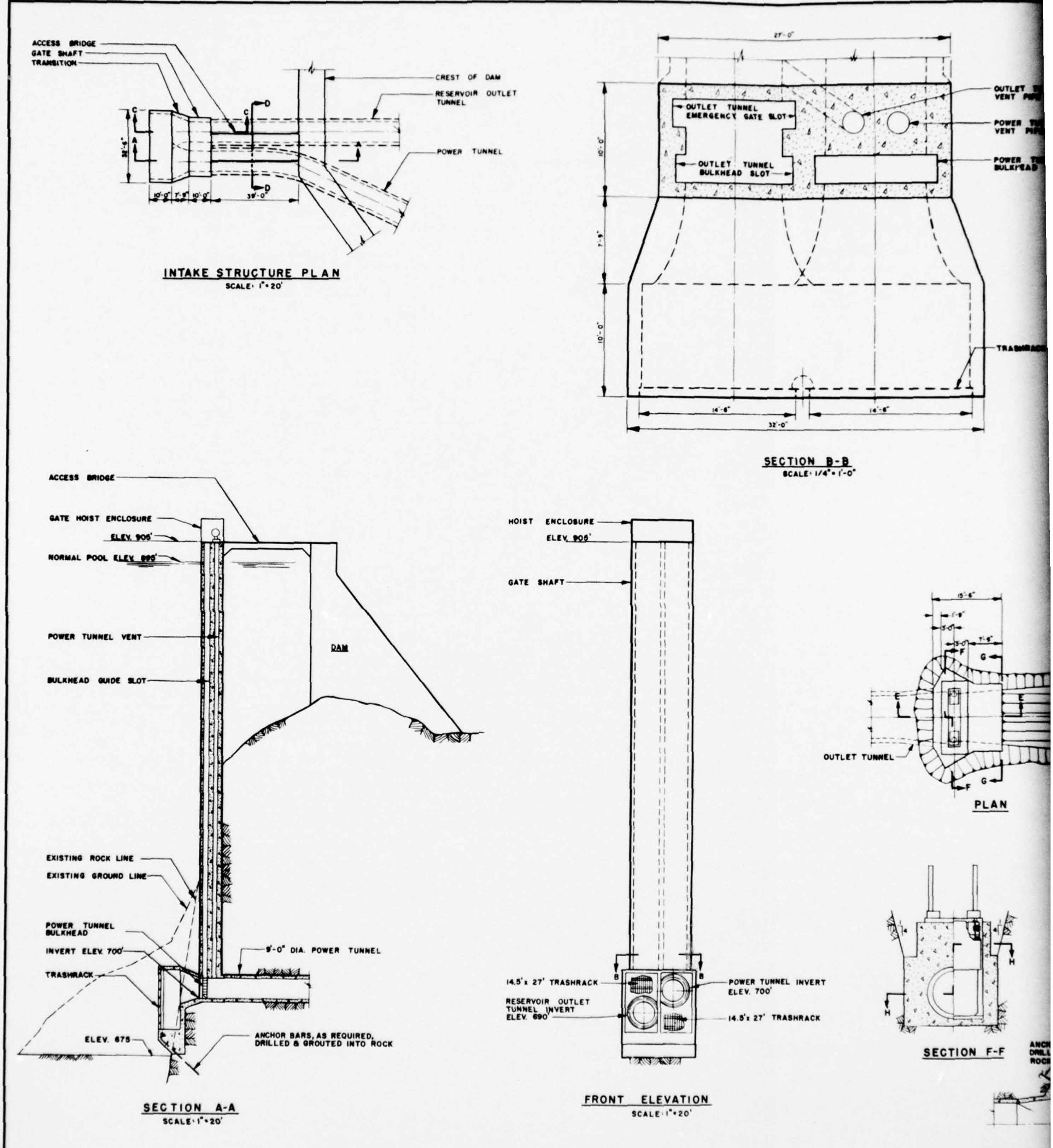


DIVERSION TUNNEL SECTIONS
SCALE 3/8"=1'-0"




GATE HOIST ENCLOSURE
ACCESS BRIDGE
MAX. NORMAL POOL EL. 895
DAM
GATE SHAFT
EXISTING ROCK LINE
EXISTING GROUND LINE
RASH RACK
NVERT EL 690'
9'-0" DIA. RESERVOIR OUTLET TUNNEL
OUTLET CONTROL STRUCTURE
INV. EL. 655
0
1
2
3
4
5
EQ.4+24 OUTLET TUNNEL = 7+58 DIVERSION TUNNEL
STATIONS
RESERVOIR OUTLET TUNNEL PROFILE
SCALE: 1" = 40'
DAM
9.5' x 10' DIVERSION TUNNEL
TUNNEL PLUG
SLOPE 0.030
OUTLET CONTROL STRUCTURE
INV. EL 555
3
4
5
6
7
8
EQ.7+58 DIVERSION = STA. 4+24 OUTLET TUNNEL
STATIONS
DIVERSION TUNNEL PROFILE
1" = 40'
GRAPHIC SCALE: 1" = 40'
GRAPHIC SCALE 1/4" = 1'-0"
REVISION
DATE
DESCRIPTION
BY
U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA
DESIGNED: WDL & JFS
DRAWN: JFS & JMR
CHECKED: WDL
PREPARED:
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
OUTLET & DIVERSION TUNNELS
SUPERVISED:
SUBMITTED:
RECOMMENDED:
APPROVED:
SCALE AS SHOWN
DATE 21 OCT. 1965
FILE NUMBER
Q-4-5-113
SHEET OF

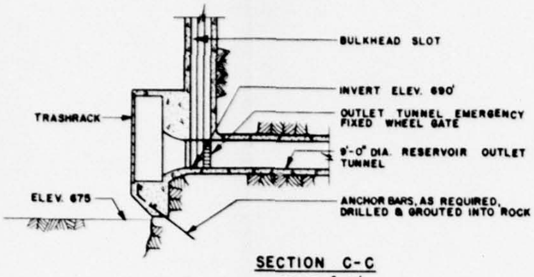
2



INTAKE STRUCTURE

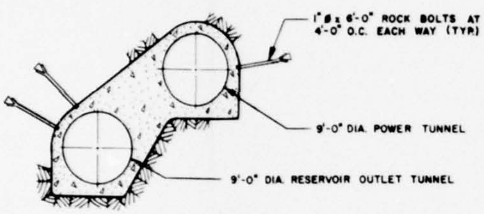
SCALE: AS SHOWN

NOTE: SEE PLATE 27 FOR LOCATIONS



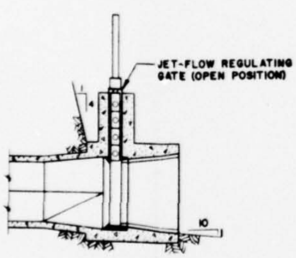
SECTION C-C

SCALE: 1" = 40'

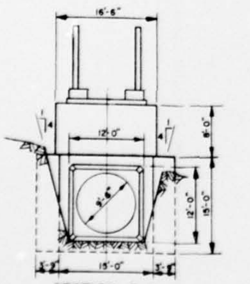


SECTION D-D

SCALE: 1/4" = 1'-0"



SECTION E-E

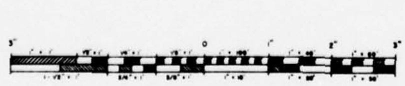


SECTION G-G

CTION H-H

TROL STRUCTURE

LE: 1" = 10'-0"



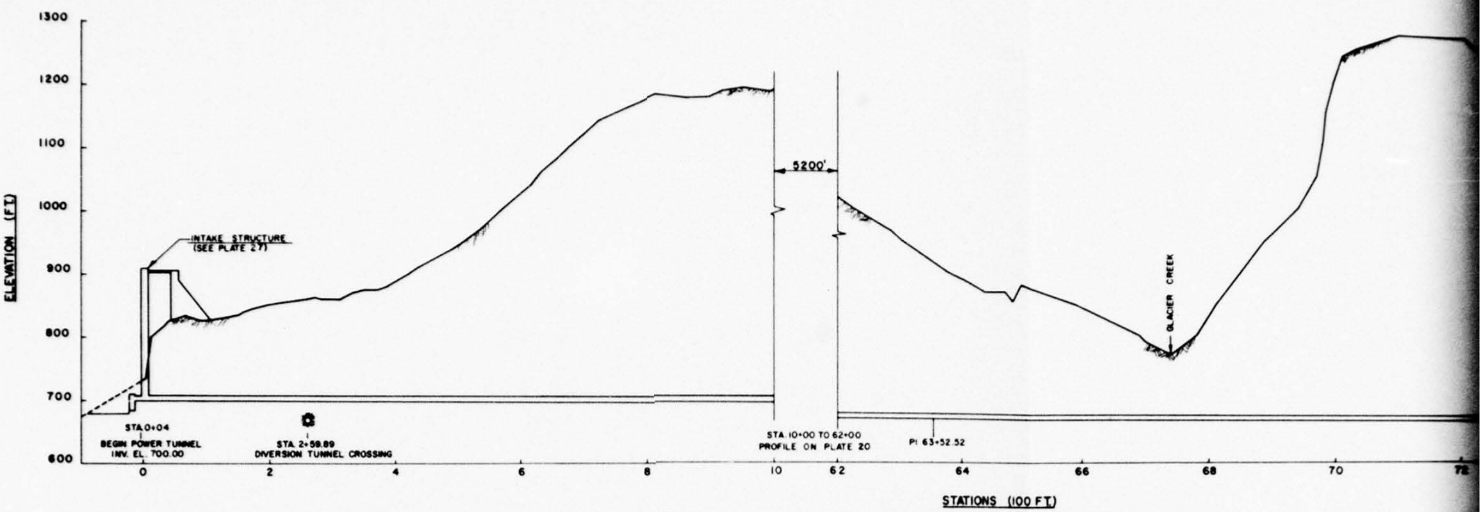
GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

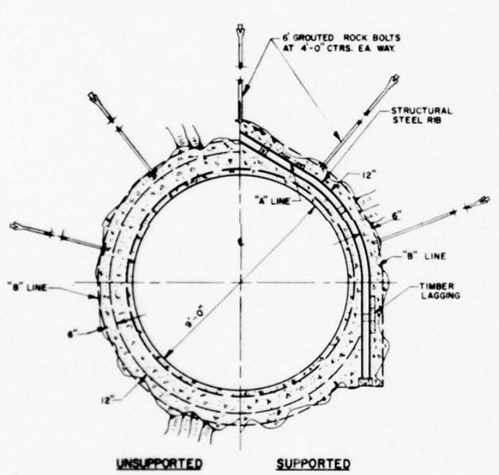
U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: J.F.S. & W.D.L.	SNETTISHAM PROJECT, ALASKA
DRAWN: J.F.S. & W.D.L.	FIRST STAGE DEVELOPMENT
CHECKED: W.D.L.	
PREPARED: W.D. [signature] CHIEF, DAM & R.R. UNIT	OUTLET & INTAKE STRUCTURES
SUPERVISED: [signature] CHIEF, INVESTIGATIONS & REPORTS SECTION	APPROVED: [signature] COLONEL, CE, DISTRICT ENGINEER
SUBMITTED: [signature] CHIEF, PLANNING & REPORTS BRANCH	SCALE AS SHOWN — DATE 21 OCT. 1965
RECOMMENDED: Warren George CHIEF, ENGINEERING DIVISION	SHEET OF — FILE NUMBER Q-4-5-113

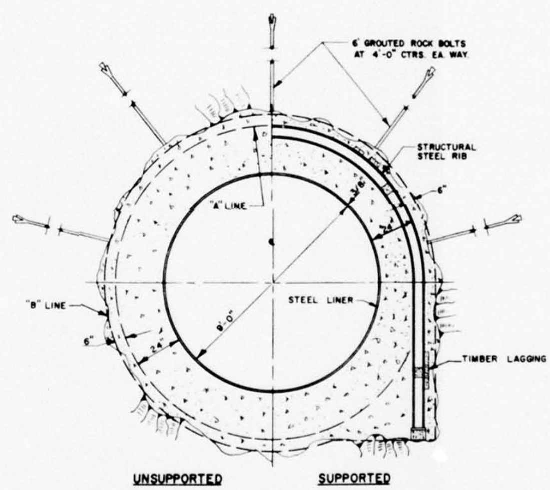
DESIGN MEMORANDUM NO. 7 PLATE 28



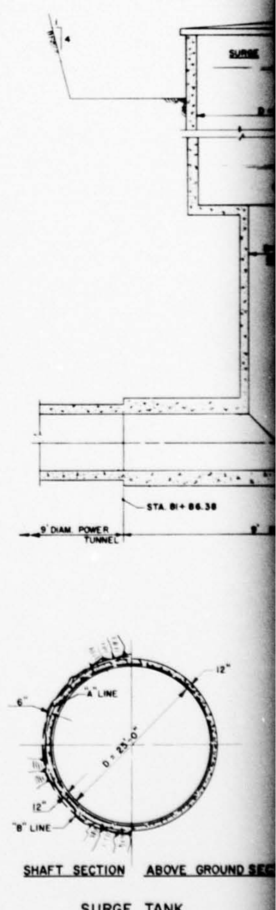
POWER TUNNEL PROFILE
SCALE: 1" = 100'



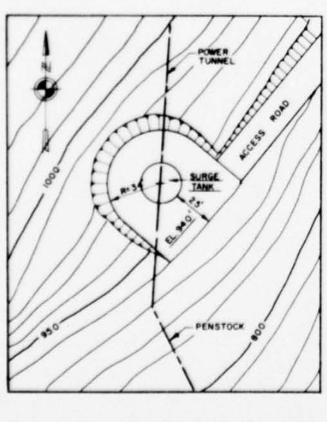
POWER TUNNEL SECTION
SCALE: 3/8" = 1'-0"



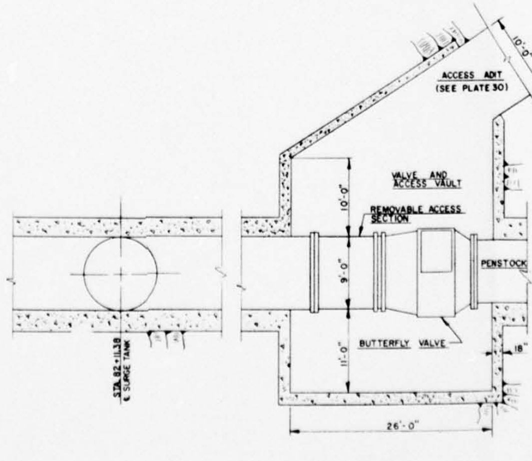
9-FT. PENSTOCK SECTION
SCALE: 3/8" = 1'-0"



SURGE TANK
SCALE: 1/8" = 1'-0"

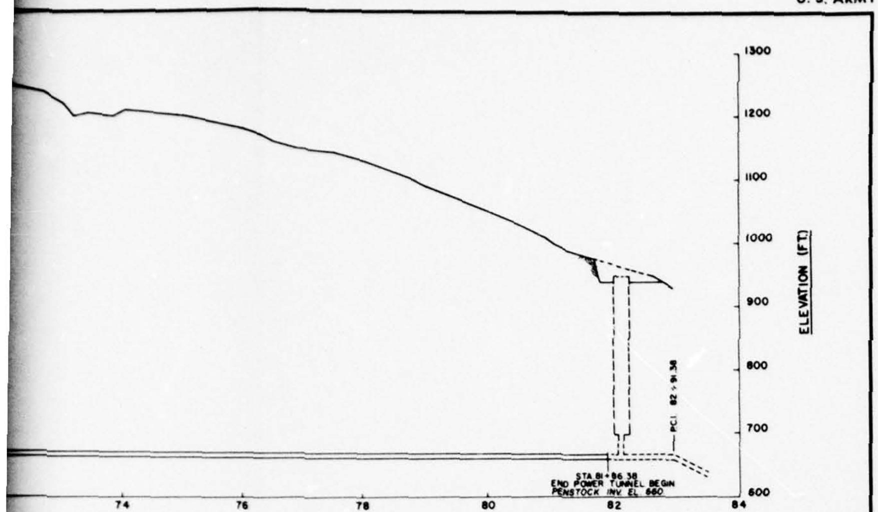


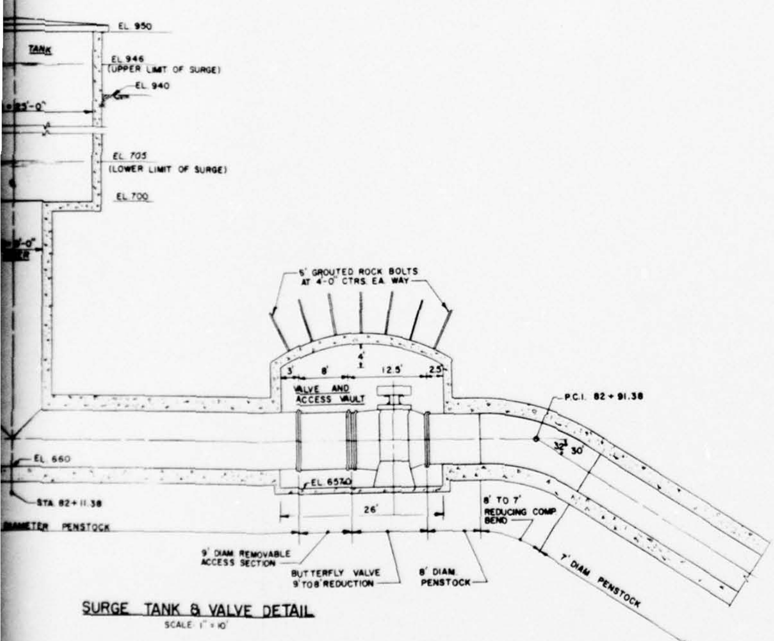
SURGE TANK GRADING PLAN
SCALE: 1" = 40'



ACCESS ADIT & VAULT DETAIL
SCALE: 1/8" = 1'-0"

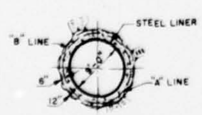
U. S. ARMY





SURGE TANK & VALVE DETAIL

SCALE: 1" = 10'



SURGE TANK RISER SECTION

SCALE: 1/8" = 1'-0"

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: L.L.L. & J.M.R.
DRAWN: J.M.R.
CHECKED: W.D.L.
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

POWER TUNNEL & SURGE TANK DETAILS

SUPERVISED: Gail J. Greenwald — CHIEF, INVESTIGATIONS & REPORTS SECTION
APPROVED: Clair F. Farley — COLONEL, CE, DISTRICT ENGINEER
SUBMITTED: — CHIEF, PLANNING & REPORTS BRANCH
RECOMMENDED: — CHIEF, ENGINEERING DIVISION

SCALE AS SHOWN | DATE 21 OCT. 1965

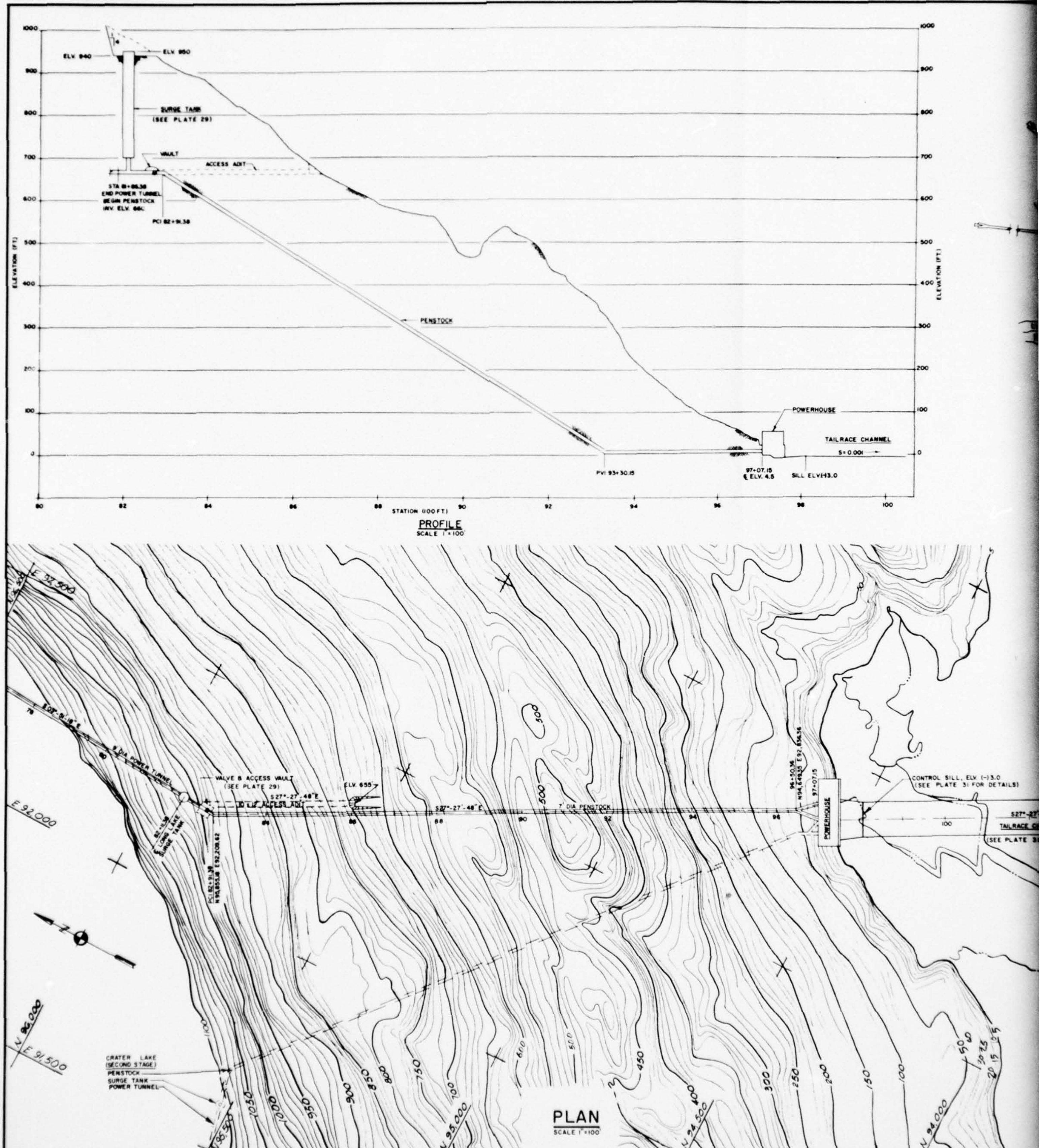
FILE NUMBER Q-4-5-113

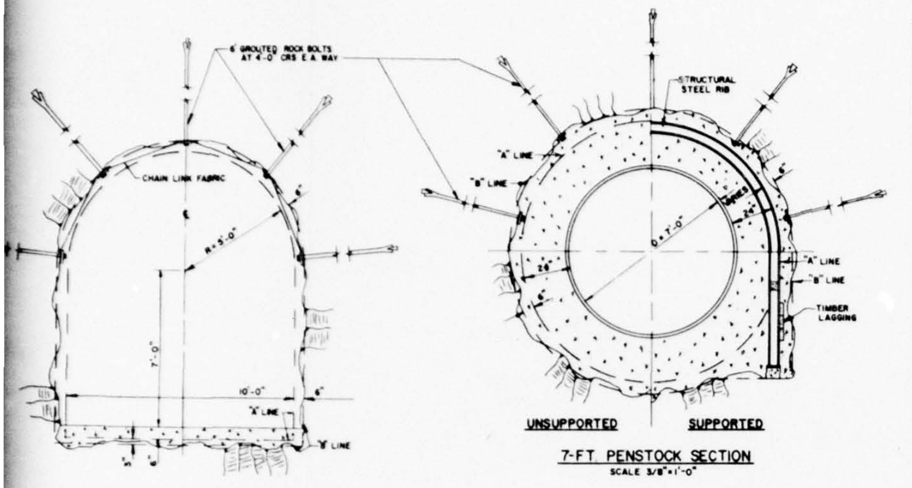
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 29

2

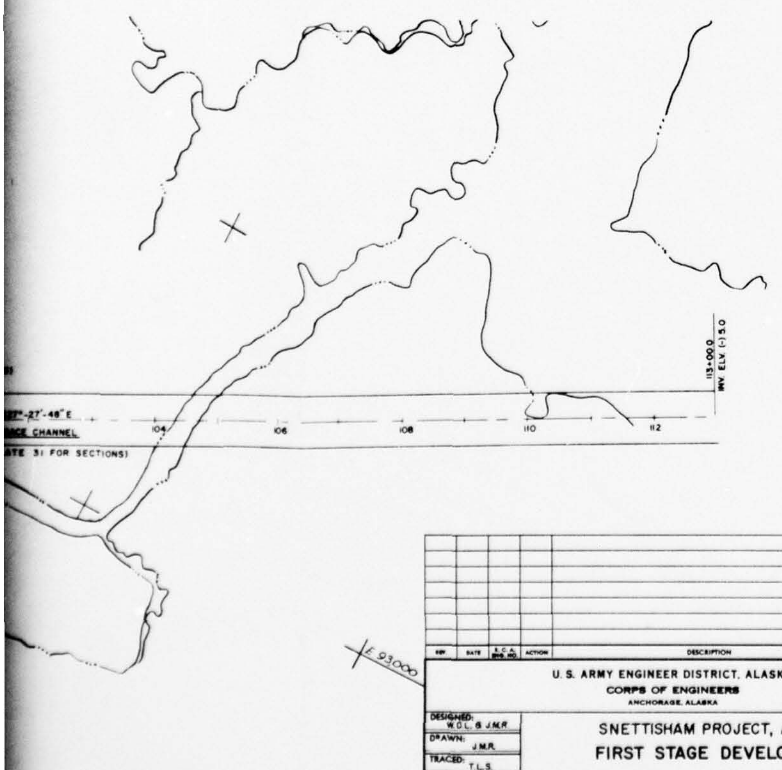
CORPS OF ENGINEERS





ACCESS ADIT SECTION
SCALE 3/8"=1'-0"

7-FT. PENSTOCK SECTION
SCALE 3/8"=1'-0"



GRAPHIC SCALE: 1" = 100'

REV.	DATE	E.C.A. ENG. NO.	ACTION	DESCRIPTION	BY	APP'D

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: W.D.L. & J.M.R.
DRAWN: J.M.R.
TRACED: T.L.S.
CHECKED: W.D.L.
SUBMITTED: CHIEF, DAM & RES. UNIT

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
PENSTOCK & TAILRACE CHANNEL

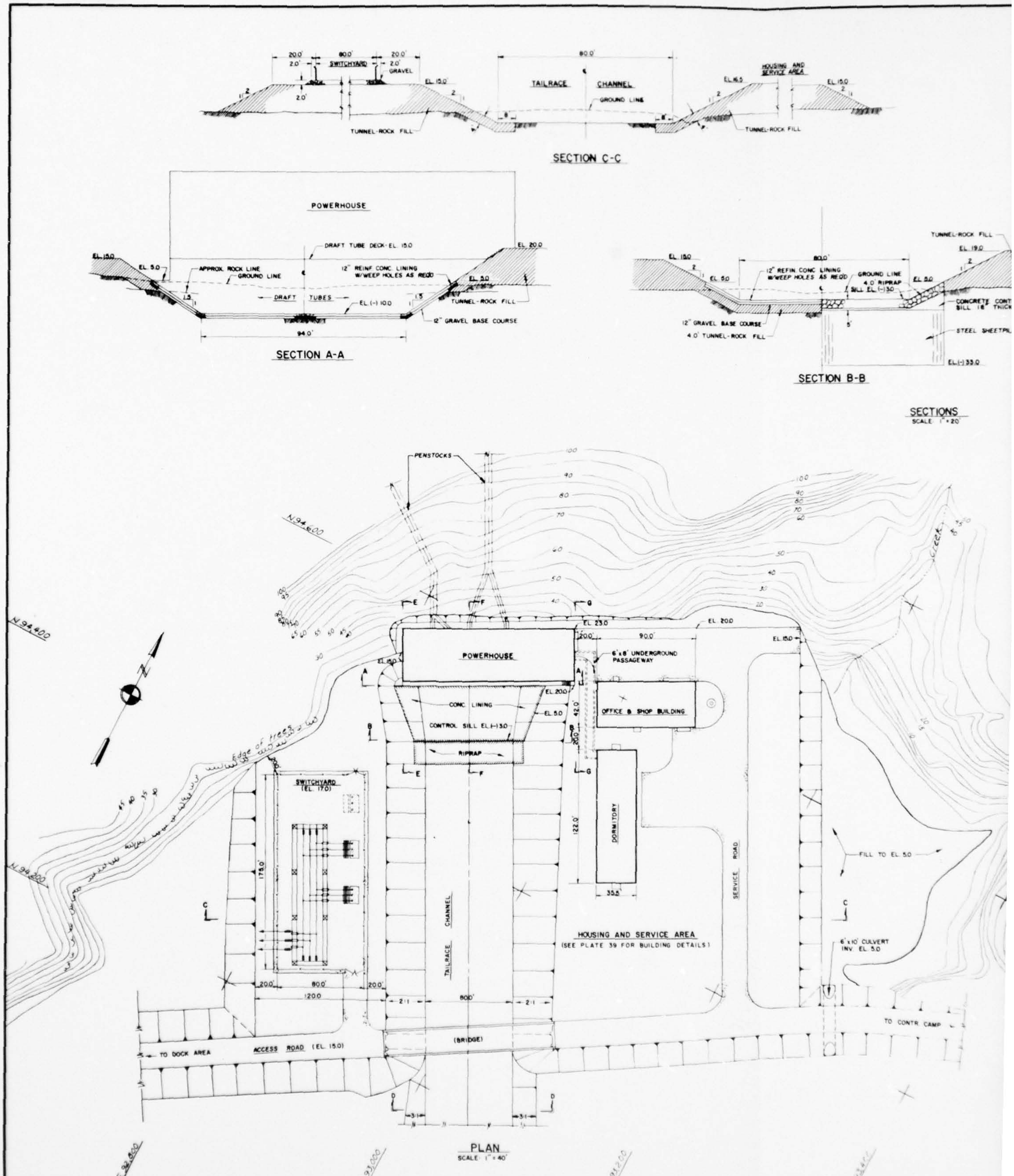
SUPERVISED: CHIEF INVESTIGATIONS & REPORTS SECTION
APPROVED: Clos. T. Tailey, COLONEL, CE, DISTRICT ENGINEER
DATE 21 OCT 1963

CHIEF PLANNING & REPORTS BRANCH
SCALE AS SHOWN
SPEC. NO.

RECOMMENDED: Warren George, CHIEF ENGINEERING DIVISION
DRAWING NUMBER Q-4-5-113
SHEET OF

DESIGN MEMORANDUM NO. 7 PLATE 30

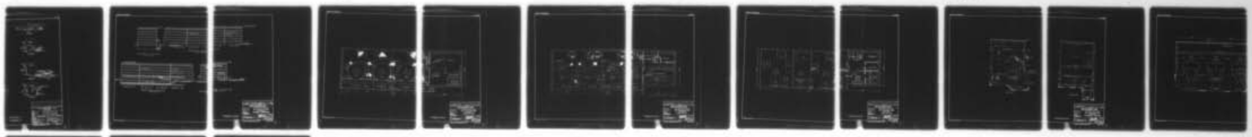
2

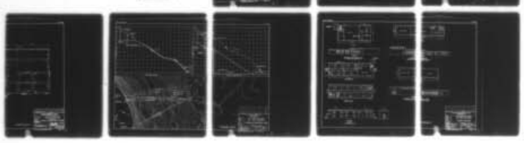

SECTION C-C
SWITCHYARD
TAILRACE CHANNEL
HOUSING AND SERVICE AREA
GROUND LINE
TUNNEL-ROCK FILL
GRAVEL
POWERHOUSE
DRAFT TUBE DECK-EL. 15.0
APPROX. ROCK LINE
GROUND LINE
12" REINF CONC LINING W/WEEP HOLES AS REQ'D
DRAFT TUBES
EL.(-)10.0
TUNNEL-ROCK FILL
12" GRAVEL BASE COURSE
94.0'
SECTION A-A
12" REFIN. CONC LINING W/WEEP HOLES AS REQ'D
GROUND LINE
4.0' RIPRAP
SILL EL.(-)13.0
CONCRETE CONT SILL 18" THICK
STEEL SHEETPIL
12" GRAVEL BASE COURSE
4.0' TUNNEL-ROCK FILL
EL.(-)33.0
SECTION B-B
SECTIONS
SCALE: 1" = 20'
PENSTOCKS
POWERHOUSE
6' x 8' UNDERGROUND PASSAGEWAY
OFFICE & SHOP BUILDING
CONC LINING
CONTROL SILL EL.(-)13.0
RIPRAP
SWITCHYARD (EL. 17.0)
DORMITORY
Edge of trees
Creek
TAILRACE CHANNEL
SERVICE ROAD
FILL TO EL. 5.0
HOUSING AND SERVICE AREA
(SEE PLATE 39 FOR BUILDING DETAILS)
6' x 10' CULVERT INV. EL. 5.0
TO CONTR. CAMP
(BRIDGE)
TO DOCK AREA
ACCESS ROAD (EL. 15.0)
PLAN
SCALE: 1" = 40'

AD-A067 894   CORPS OF ENGINEERS ANCHORAGE AK ALASKA DISTRICT   F/G 13/12
SNETTISHAM PROJECT ALASKA. FIRST STAGE DEVELOPMENT. DESIGN MEMO--ETC(U)
OCT 65

UNCLASSIFIED   NL

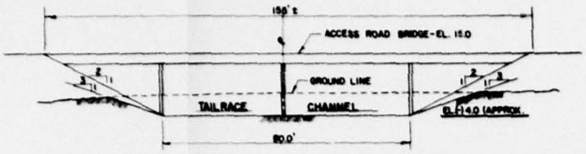
4 OF 4
AD
A067894



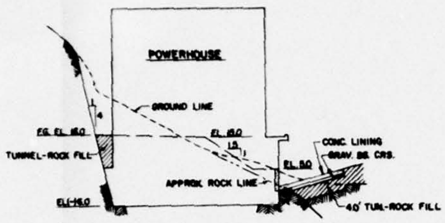


END
DATE
FILMED
6 --79
DDC

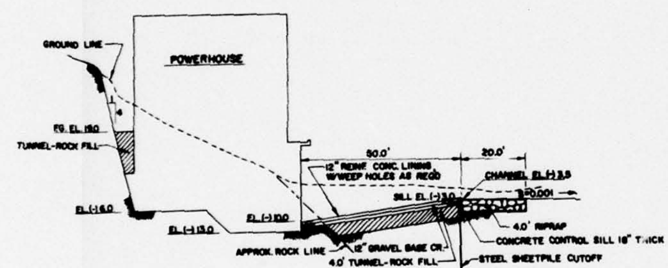
U. S. ARMY



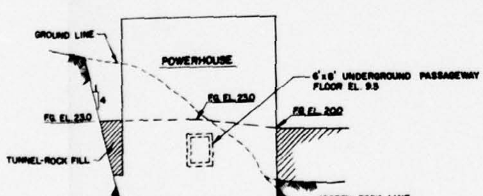
SECTION D-D



SECTION E-E



SECTION F-F



SECTION G-G

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: W.D.L.
DRAWN: W.D.L. & V.H.
CHECKED: W.D.L.
PREPARED: W. D. Lloyd

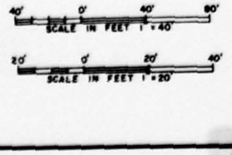
SNETTISHAM PROJECT, ALASKA

FIRST STAGE DEVELOPMENT

POWERHOUSE SITE PLAN & DETAILS

SUPERVISED: Carl J. Gronewald
SUBMITTED: [signature]
RECOMMENDED: Marcus George
APPROVED: [signature]

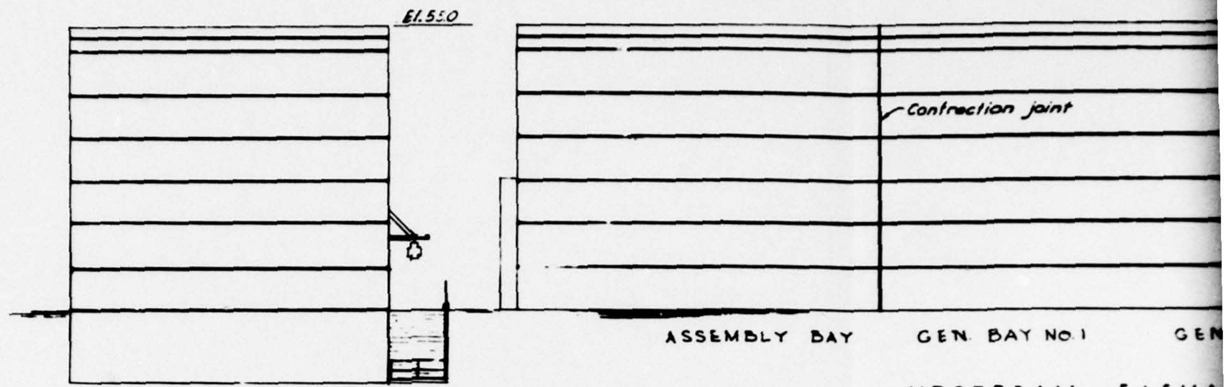
SCALE AS SHOWN | DATE 21 OCT 1965
FILE NUMBER Q-4-5-113
SHEET OF



DESIGN MEMORANDUM NO. 7 PLATE 31

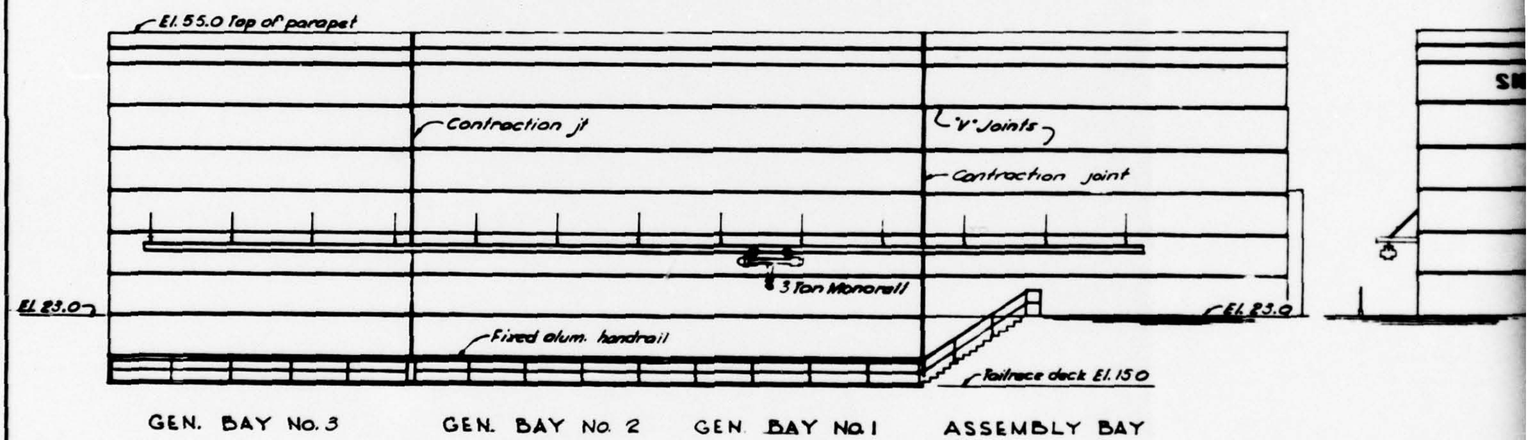
2

CORPS OF ENGINEERS



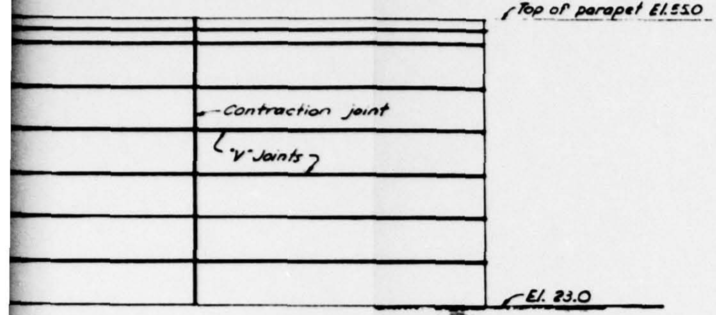
UPSTREAM ELEVA
Scale 1/8" = 1'-0"

SOUTHWEST ELEVATION
Scale: 1/8" = 1'-0"



DOWNSTREAM ELEVATION
Scale: 1/8" = 1'-0"

NORTHE



ATION

EAST ELEVATION
Scale: 1/8" = 1'-0"

GRAPHIC SCALE: 1/8" = 1'-0"

Revision	Date	Description
1	10 Oct 68	Increased length of Assembly Bay

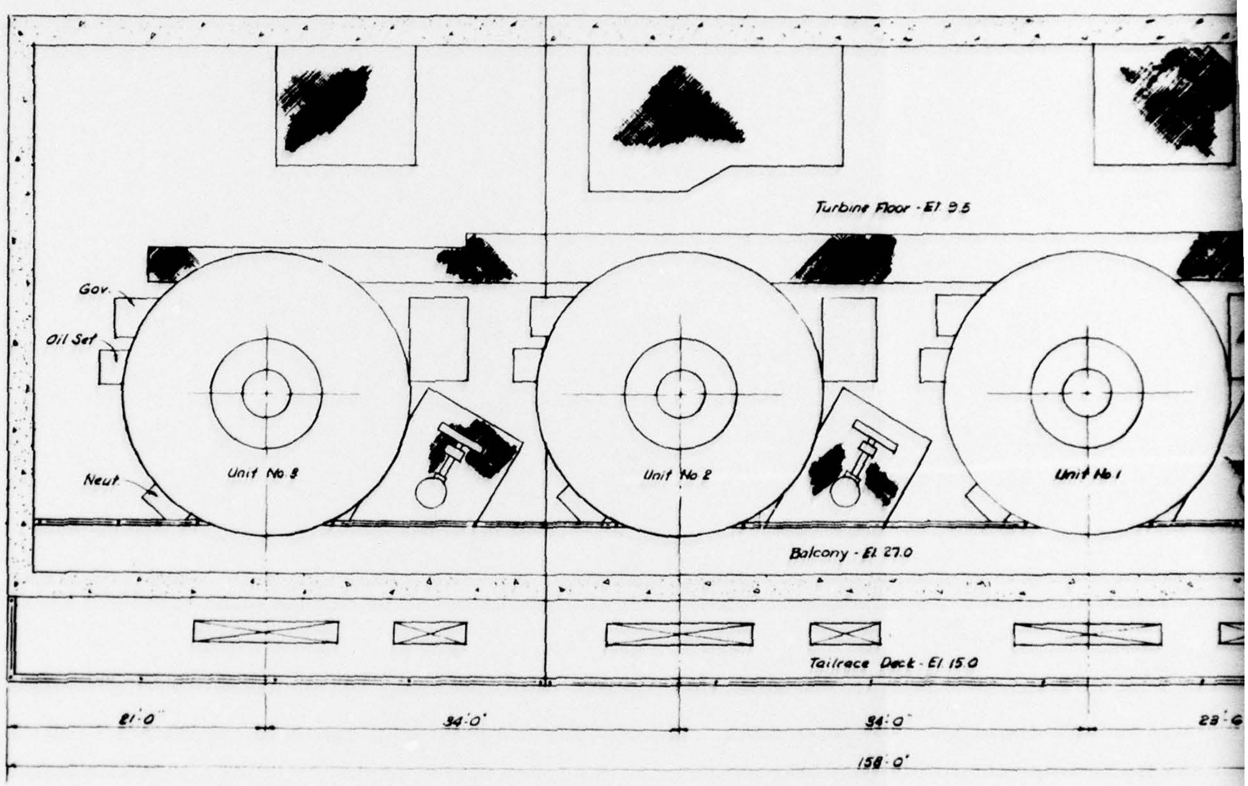
CORPS OF ENGINEERS, U. S. ARMY
NORTH PACIFIC DIVISION, PORTLAND, OREGON

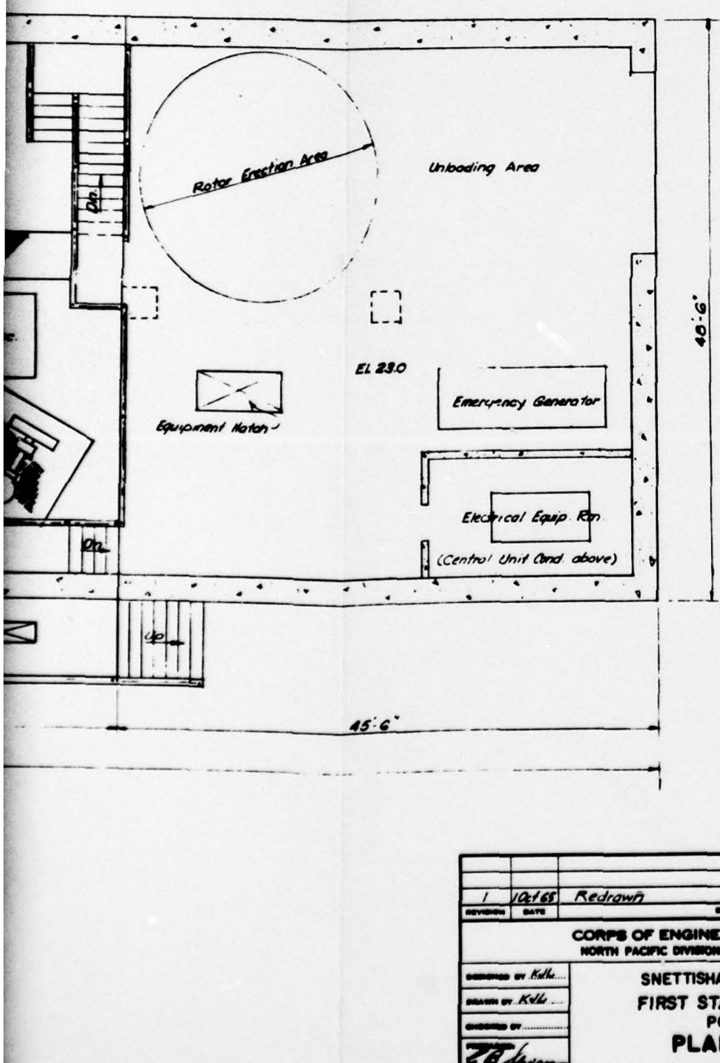
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
POWERHOUSE
EXTERIOR ELEVATIONS

SNP 0-0-0/1

DESIGN MEMORANDUM NO. 7 PLATE 32

2


Turbine Floor - El. 9.5
Gov.
Oil Set
Neut.
Unit No 3
Unit No 2
Unit No 1
Balcony - El. 27.0
Tailrace Deck - El. 15.0
21'-0"
34'-0"
34'-0"
158'-0"



REVISION	DATE	DESCRIPTION	BY
1	1 Oct 65	Redrawn	

CORPS OF ENGINEERS, U. S. ARMY
NORTH PACIFIC DIVISION, PORTLAND, OREGON

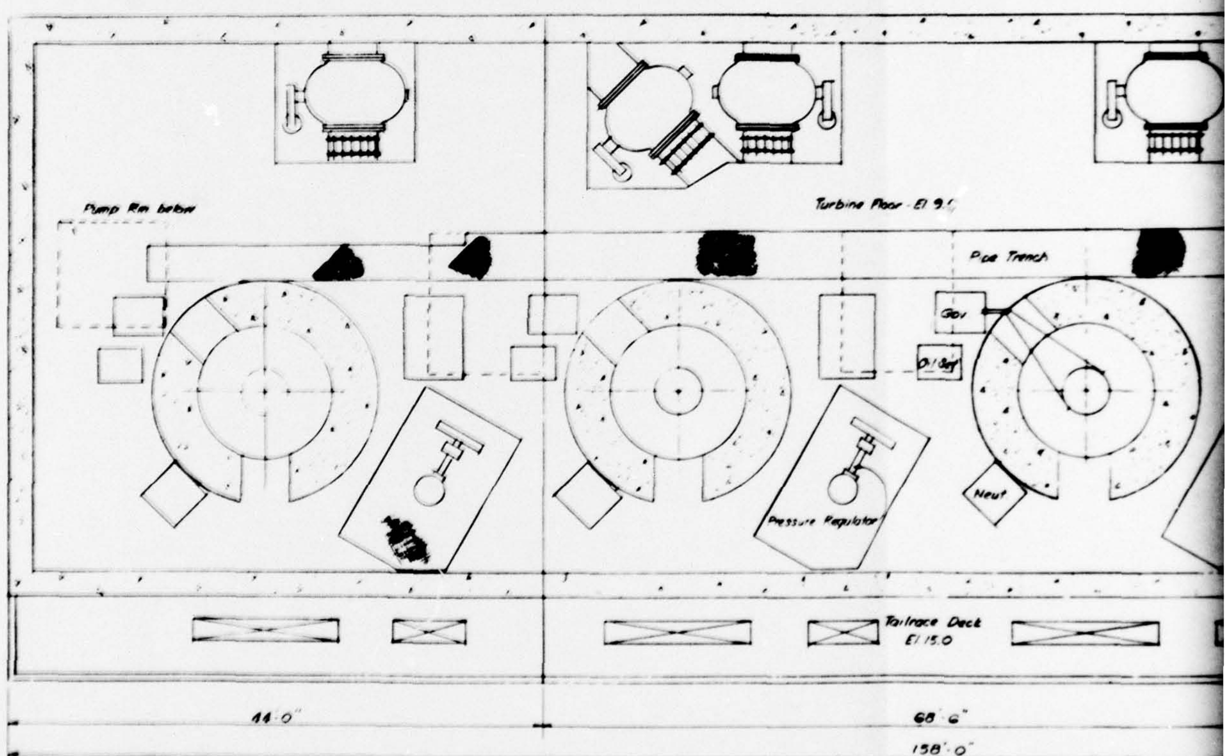
DESIGNED BY Kdh
DRAWN BY Kdh
CHECKED BY

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
POWERHOUSE
PLAN - EL. 23.0

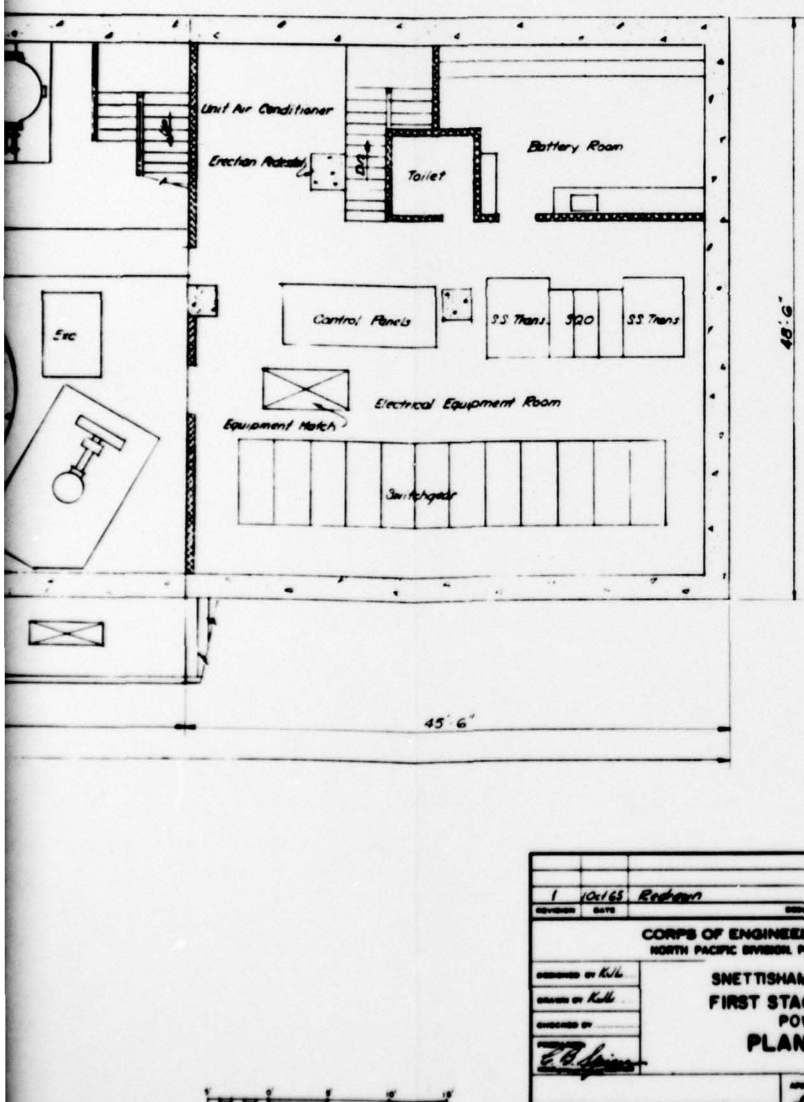
DATE 1 Oct 1965
SCALE 3/16" = 1'-0"
SPEC. NO.

SNP-0-0-0/2

CORPS OF ENGINEERS



U. S. ARMY



Revision	Date	Description	By
1	1Oct65	Redrawn	

CORPS OF ENGINEERS, U. S. ARMY
NORTH PACIFIC DIVISION, PORTLAND, OREGON

Designed by KJL
Drawn by KJL
Checked by

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
POWERHOUSE
PLAN - EL. 9.5

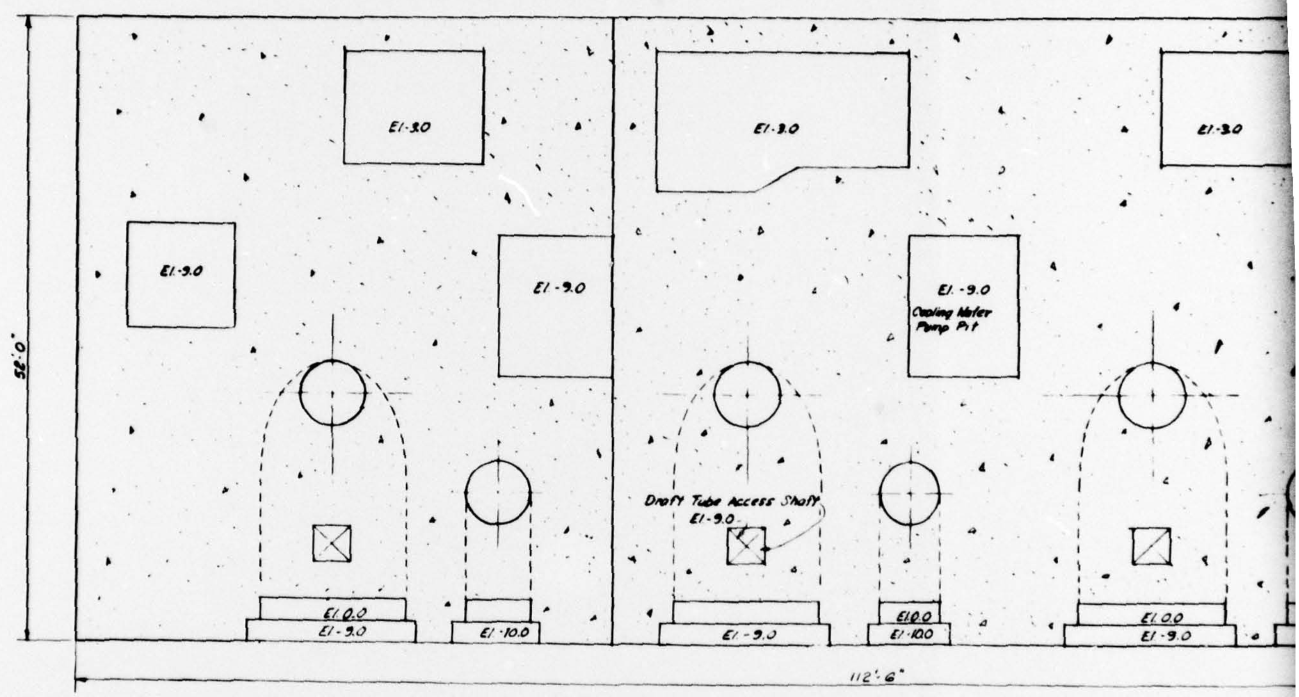
Date 1 Oct. 1965
Scale 3/16" = 1'-0"
Spec. No.

SNP 0-0-0/3

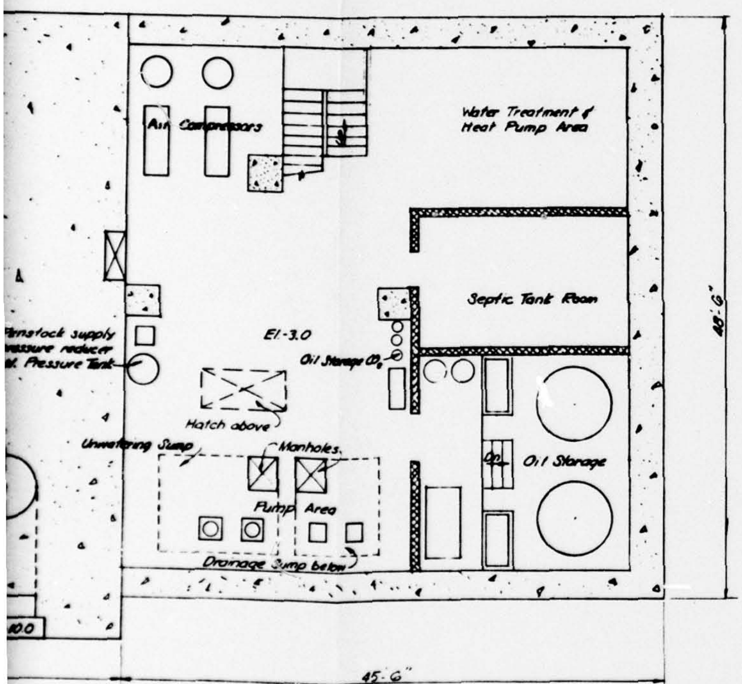
DESIGN MEMORANDUM NO. 7 PLATE 34

2

CORPS OF ENGINEERS



U. S. ARMY



GRAPHIC SCALE: 5/16" = 1'-0"

REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
NORTH PACIFIC DIVISION, PORTLAND, OREGON

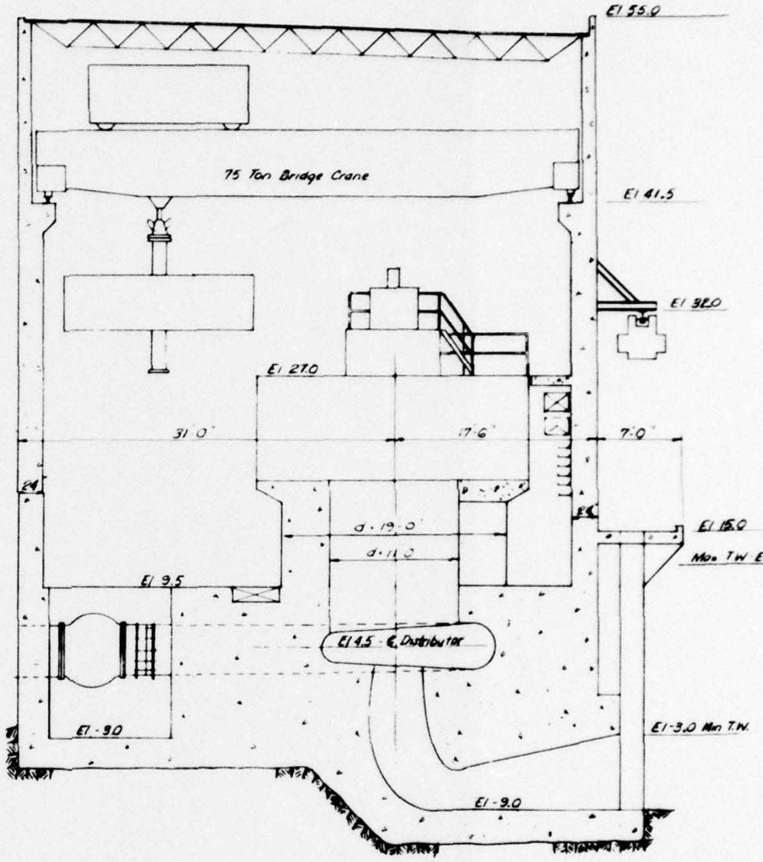
DESIGNED BY KJL
DRAWN BY KJL
CHECKED BY
PREPARED

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
POWERHOUSE
PLAN - EL.-3.0

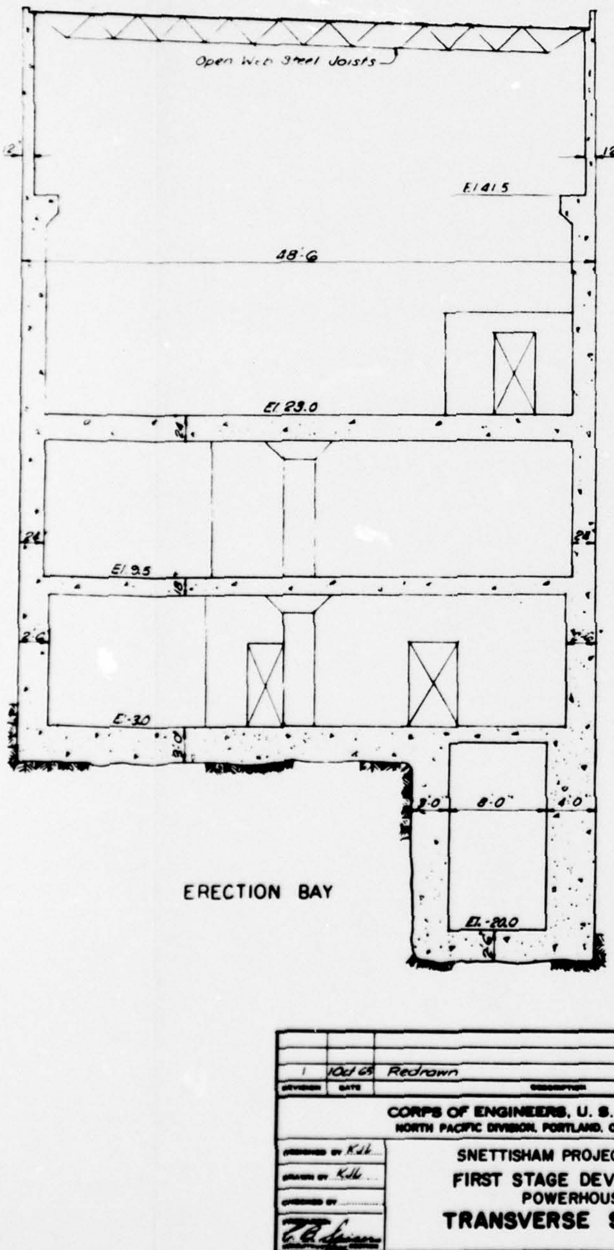
SUBMITTED
APPROVED FOR DIV. ENGINEER DATE 1 Oct. '68
SCALE 5/16" = 1'-0" SPEC. NO.
SNP 0-0-0/11

DESIGN MEMORANDUM NO. 7 PLATE 35

CORPS OF ENGINEERS



MAIN UNIT



GRAPHIC SCALE: 3/16" = 1'-0"

1	1Oct 65	Redrawn	
REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
NORTH PACIFIC DIVISION, PORTLAND, OREGON

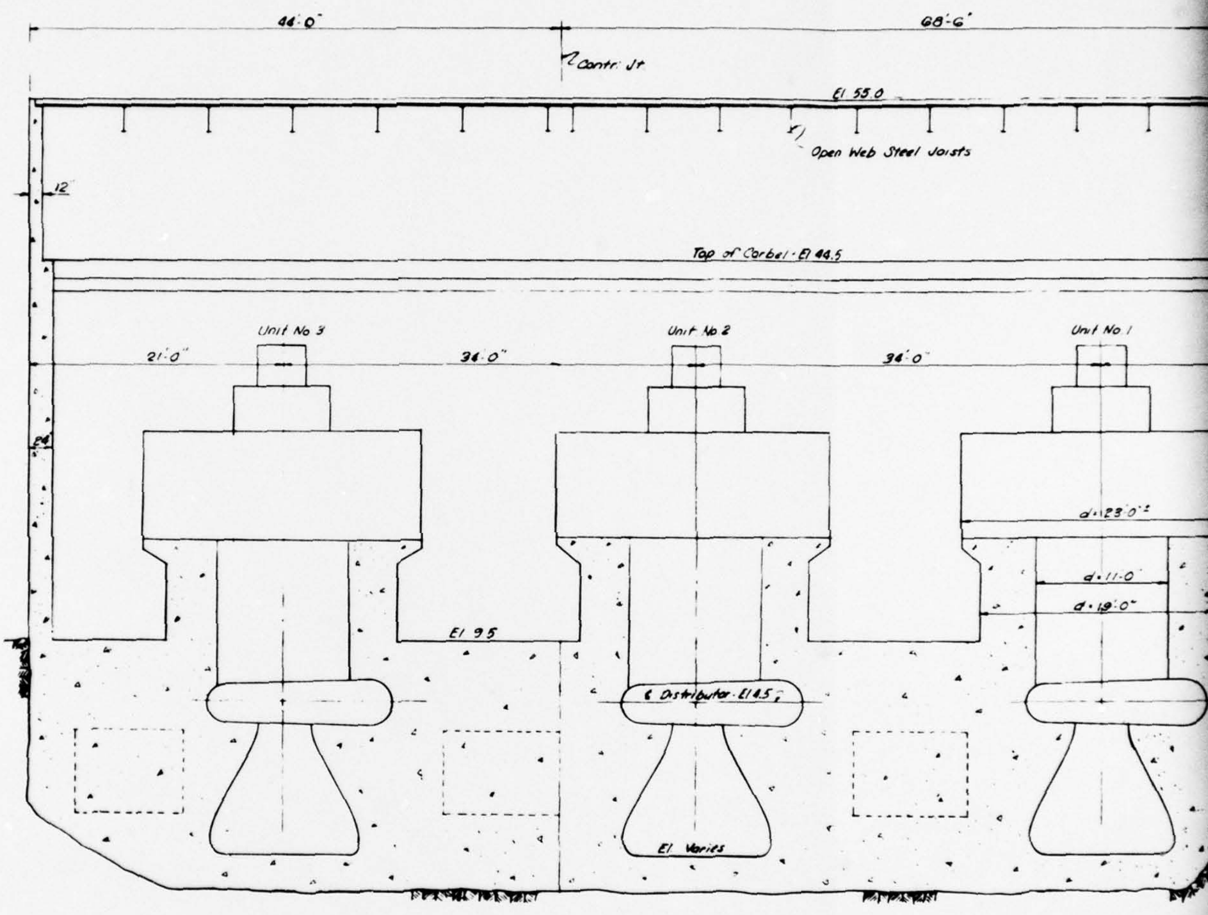
DESIGNED BY KJL
DRAWN BY KJL
CHECKED BY

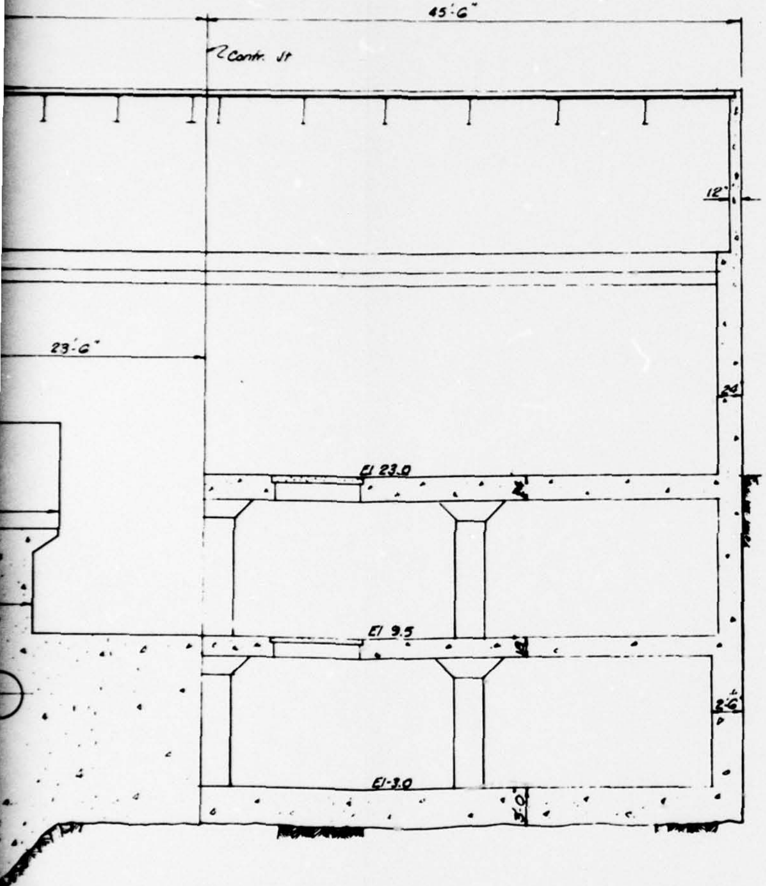
SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
POWERHOUSE
TRANSVERSE SECTIONS

DATE 1 Oct '65
SCALE 3/16" = 1'-0"
SPEC. NO.

SNP 0-0-0/4

CORPS OF ENGINEERS





1	1 Oct 65	Redrawn	
REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
NORTH PACIFIC DIVISION, PORTLAND, OREGON

DESIGNED BY Kdle
DRAWN BY Kdle
CHECKED BY

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
POWERHOUSE
LONGIT. SECTION THRU ℄ UNITS

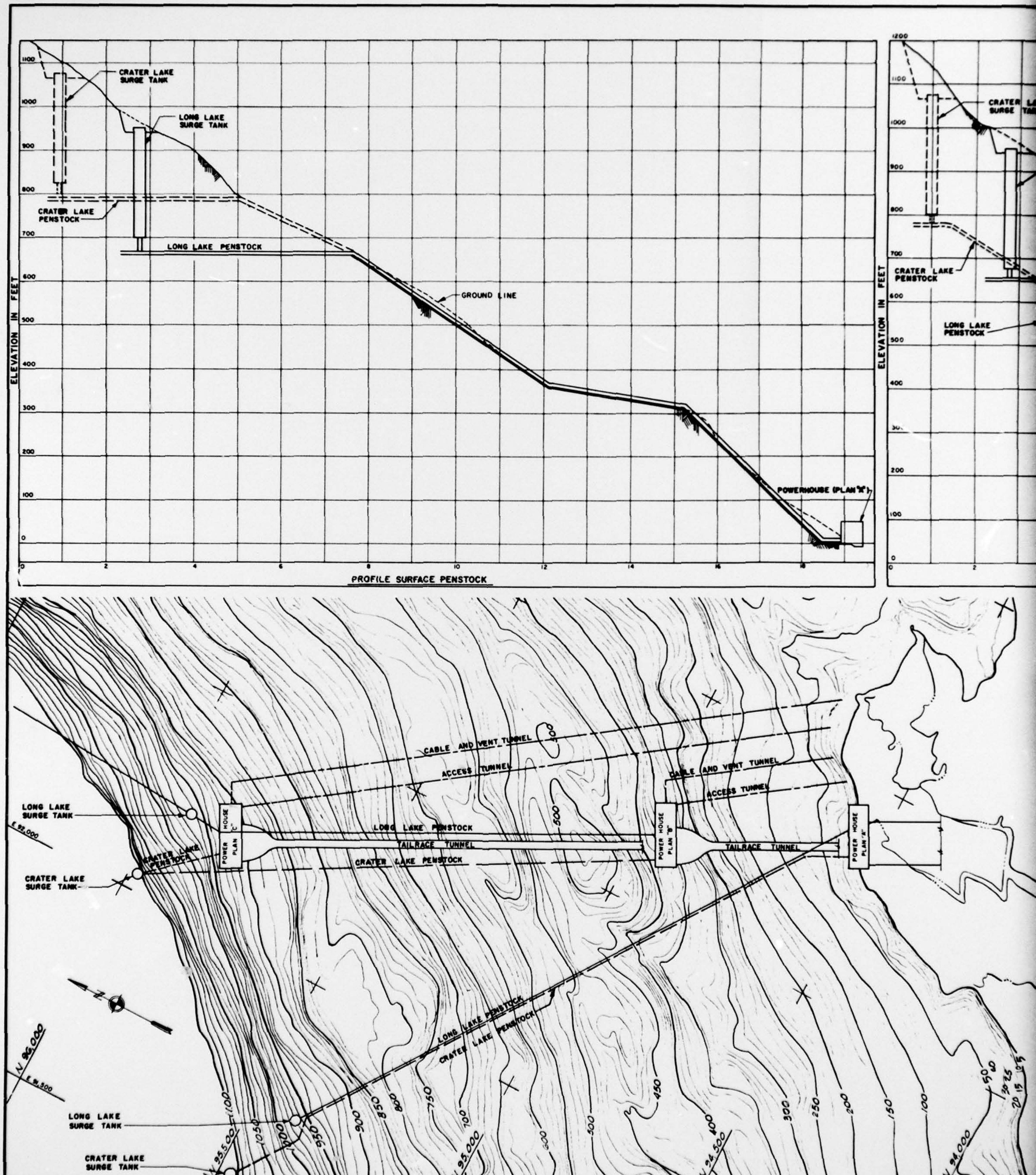
SCALE 1/16" = 1'-0"

SNP 0-0-0/5

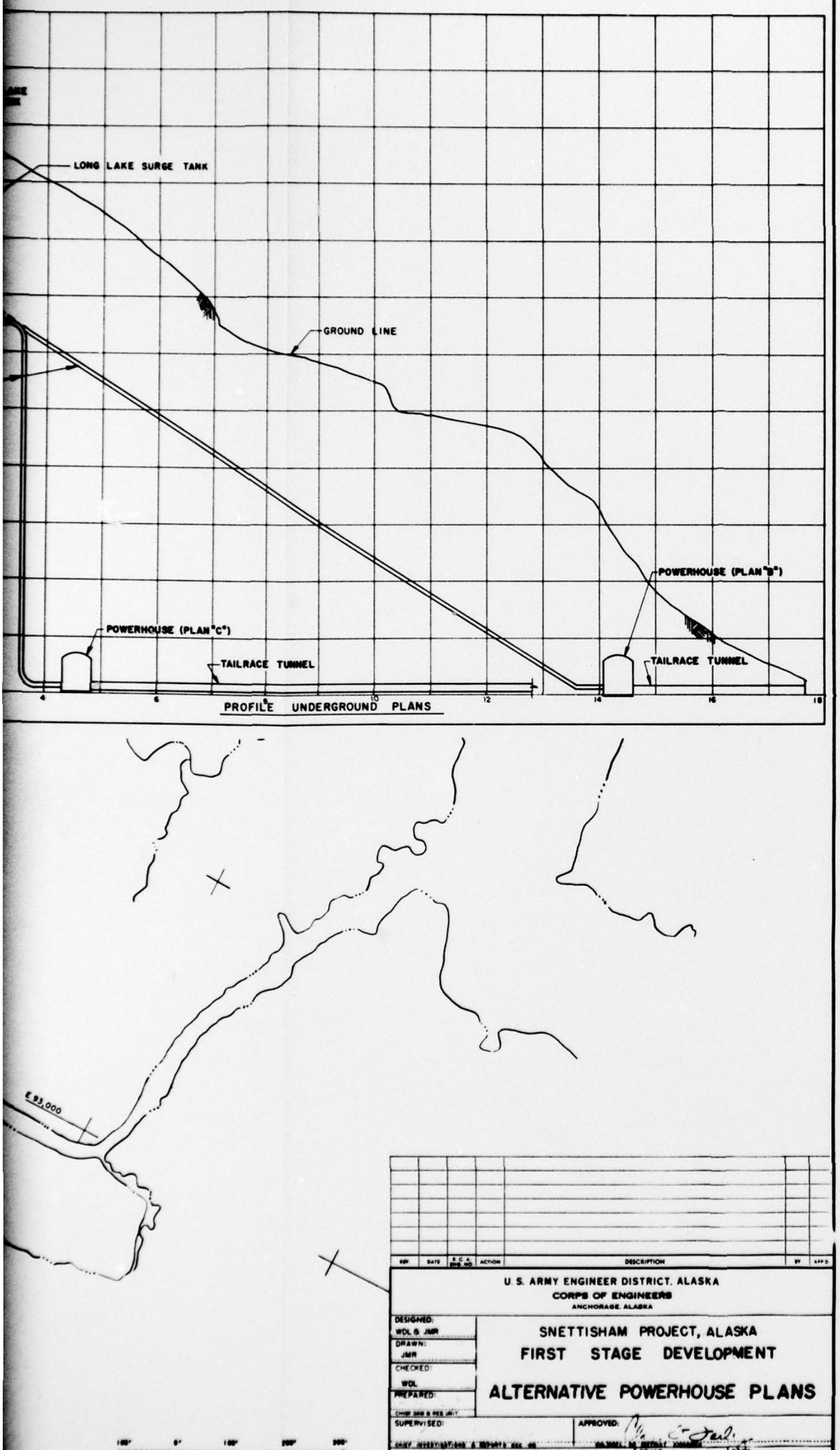
GRAPHIC SCALE: 3/16" = 1'-0"

2

CORPS OF ENGINEERS



U. S. ARMY



REV	DATE	E.C.A. DWG. NO	ACTION	DESCRIPTION	BY	APP'D

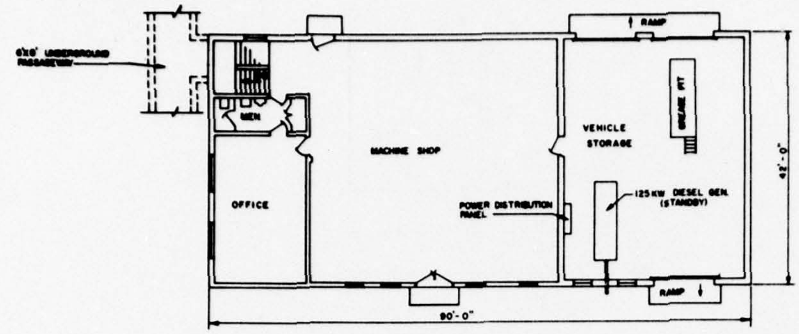
U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: WDL & JMR
DRAWN: JMR
CHECKED: WDL
PREPARED:

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT
ALTERNATIVE POWERHOUSE PLANS

SUPERVISED:
APPROVED:
SUBMITTED:
RECOMMENDED:
SCALE 1"=100'
DATE 22 OCT 1965
FILE NUMBER Q 4-5-113
SHEET OF

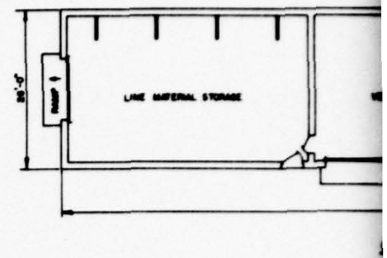
DESIGN MEMORANDUM NO. 7 PLATE 38

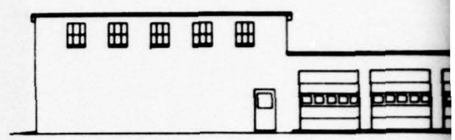


PLAN

ELEVATIONS

POWERHOUSE OFFICE & SHOP BUILDING
(SEE PLATE 31 FOR LOCATION)





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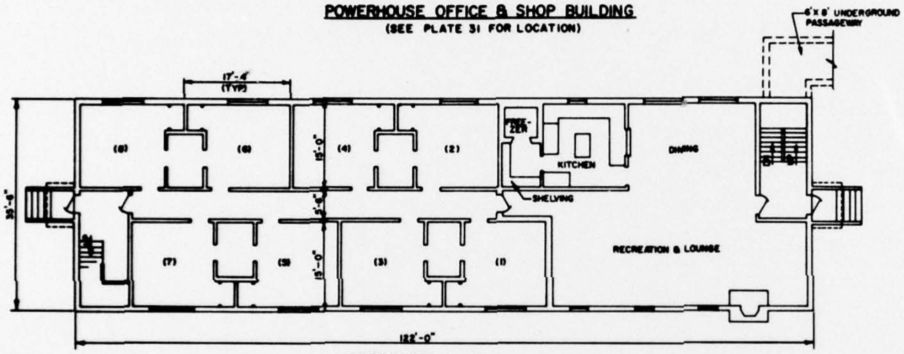
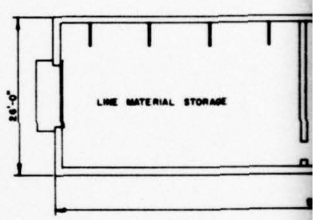
CONTROL & ST



FIRST FLOOR PLAN



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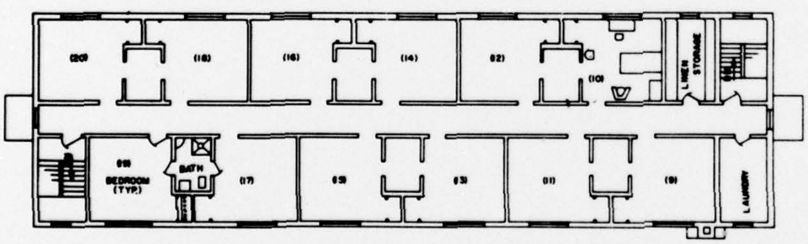
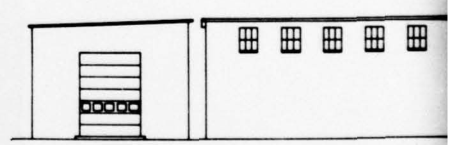
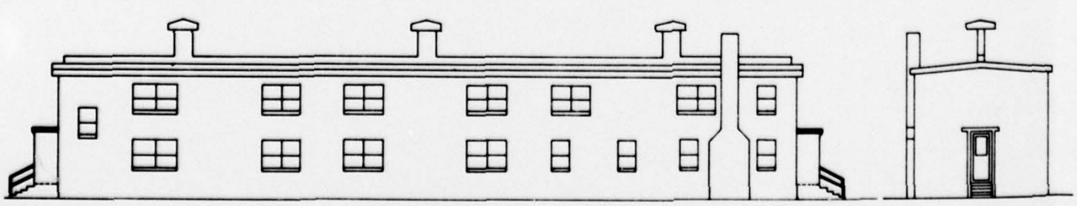


SECOND FLOOR PLAN



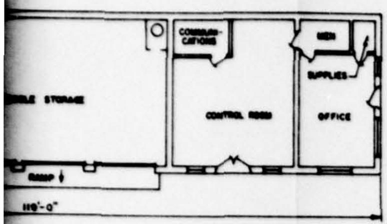
ELEVATI

TRANSMISSION MAINT
(SEE PLATE 3 FO



ELEVATIONS

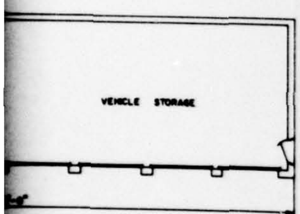
DORMITORY
(SEE PLATE 31 FOR LOCATION)



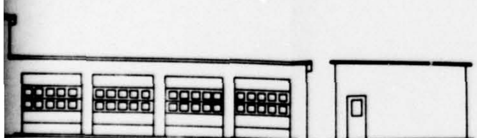
PLAN

ATIONS

SUBSTATION
ORAGE BUILDING



AN



NANCE BUILDING
APPROX. LOCATION)

U. S. ARMY ENGINEER DISTRICT, ALASKA
CORPS OF ENGINEERS
ANCHORAGE, ALASKA

DESIGNED: C.L.K.
DRAWN: C.L.K.
CHECKED: W.D.L.

SNETTISHAM PROJECT, ALASKA
FIRST STAGE DEVELOPMENT

SUPPORT BUILDINGS

SCALE 3/32" = 1'-0"
DATE 21 OCT. 1968
FILE NUMBER Q-4-5-113

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